JOURNAL OF
THE INSTITUTION OF
CIVIL ENGINEERS.

ORDINARY MEETING.

25 January, 1938.

WILLIAM JAMES EAMES BINNIE, Vice-President,
in the Chair.

The Council reported that they had recently transferred to the class of

Members.

LIONEL BAYARD AYLEN, A.F.C.

FRANK LONGDEN BRONSTORPH, M.Sc. (*Bristol*).

EDMUND GRAHAM CLARK, M.C., B.Sc. (*Durham*) (Secretary Inst. C.E.).

EDGAR ALGERNON CROSS, B.Sc. (*Birmingham*).

HAROLD FIRTH.

ALFRED PROOM HUMBLE.

JOHN PROWETT LE GRAND.

SUPPRAMANIAM MAHADEVA.

DUDLEY HEPBURN STENT.

GEORGE ERIC TAYLOR, B.Sc. (*Manchester*).

And had admitted as

Students.

PETER LAWRIE AITKEN.

RICHARD ARUNDELL.

JOHN ANTHONY BAINES, B.Eng. (*Sheffield*).

JOHN LLOYD BANNISTER, B.Sc. Tech. (*Manchester*).

ROBERT PEARMAN BEATON.

NORMAN BIRCH.

ALEXANDER DONALD BLACKIE.

HARRY BOARDMAN, B.Sc. Tech. (*Manchester*).

COLIN DONALD BRERETON, B.Sc. (Eng.) (*Lond.*).

THOMAS GEORGE BRIERLEY.

LEONARD GERALD BROADBERRY.

GEORGE RICHARD BUSH.

CHARLES CYRIL BYROM.

JOSEPH REGINALD CAPO-BIANCO.

ALAN CHANCE.

RICHARD HUGH COGSWELL.

HENRY ARCHIBALD COLES, B.Sc. (*Birmingham*).

NEVILLE THOMAS COOPER.

RABINDRA CHANDRA DAS GUPTA, B.C.E. (*Patna*).

SAVUNDRANAYAGAM ALFRED EDWARDS.

NORMAN GEORGE EGGLETON.

RICHARD GEORGE STEVENS FARRANTS.

OLADIPO ROTIMI IBIKUNLE GEORGE.

DOUGLAS HAROLD GLOVER.

ALAN GRAHAM GOWERS.

CHARLES HUGH GREENWOOD.

Donald Benbow Griffiths.
Sami Ibrahim Haim.
Arthur Andreas Southern Hammer.
John David Hardie.
Reginald Walter John Hewson.
Eric Sanderson Hill.
Edward John Hobbs.
William Derek Holland
Gerald Herbert Holloway, B.Sc. (Edin.).
William Bruce Keith.
George Matthews Lewis, B.A. (Cantab.).
Peter Renell Lisle.
Norman Ragnar Logsdon.
Kenneth Stanford London.
Edmund Peter McBretney.
John McGee.
James Kenneth McIntyre.
Leonard John Marcus McSweeney.
James Bushe Manson.
Neil Malcolm Rowland Moody.
William George Mosses, B.Eng. (Sheffield).
Howard Donald Alfred Murdoch.
Thampu Mylvaganam.
Robert Henderson Ogilvie Northcote, B.Sc. (Edin.).

John Echlin O'Donnell.
Yashraj Govind Patel.
Hyam Solomon Reuben.
Dennis Gilbert Riley.
Denis Davison Roberts.
Ronald Henry Fletcher Roe.
Stephen Cowell Saynor.
Arthur Rees Schroder.
George Selvadurai Sinnatamby.
John Allen Sopp, B.Sc. (Durham).
George Mitchell Stelfox.
Benjamin Charles Graham Stevens, B.A. (Cantab.).
Basil Ernest Thompson.
George Maughan Thompson, B.Sc. (Eng.) (Lond.).
John Thomson.
Robert Haigh Walker.
Henry William Walter.
John Brian Walton, B.Sc. (Eng.) (Lond.).
Leslie Cartwright Whalley.
Neil Whiston.
Harold Ellis Whyte, B.Sc. (St. Andrews).
William Kenneth Wintersgill.
Leslie Ernest Wyatt, B.Sc. (Bristol).

The following Paper was submitted for discussion, and, on the motion of the Chairman, the thanks of The Institution were accorded to the Author.

Paper No. 5121.

"The Subsidence of a Rockfill Dam and the Remedial Measures employed at Eildon Reservoir, Australia." †

By RUPERT GRENVILLE KNIGHT, M.C., M.C.E., M. Inst. C.E.

(*Abridged*)

TABLE OF CONTENTS.

INTRODUCTION.

THE Eildon reservoir, the chief storage of the Goulburn—Waranga—Mallee irrigation, stock- and domestic-supply system, is the largest project of this nature within the State of Victoria, and its value and importance in the economic life of the community have steadily increased in recent years.

Constructed on the Goulburn river in the rugged country about 90 miles to the north-east of Melbourne, the Eildon dam, which is of the rockfill type, was completed in 1927; it partially failed in April 1929, and during the next 7 years remedial measures and improvements costing £380,000 were carried out. This Paper describes the work entailed in the restoration of the embankment, the drainage of the foundations, and the remodelling of the outlet-works and spillway.

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† Correspondence on this Paper can be accepted until the 1st July, 1938. —SEC. INST. C.E.

Geographical and Historical.

Eastern and southern Victoria are within a belt of ample rainfall of from 20 inches to 60 inches per annum, but semi-arid conditions prevail in the north-west of Victoria, and occupation is rendered " safe " only in so far as productivity is sustained by water-conservation and irrigation.   Successful settlement thus depends on the more intense development of those areas with a rainfall of 20 inches or more, and the removal of the risks attendant upon the occupation of the semi-arid areas by the adoption of irrigation.

Irrigation had been discussed in Victoria since about 1840[1] and, following the drought of 1877–1881, the Victorian Government took the first decisive steps.

Successive Water-Conservation and Irrigation Acts were passed, which led finally to the establishment of the State Rivers and Water Supply Commission, comprising three Commissioners, in which was vested the entire management of the irrigation-areas.

THE GOULBURN SYSTEM.

The oldest and largest irrigation-scheme in Victoria, the Goulburn system, was inaugurated by the construction between 1887 and 1891 of the Goulburn weir, near Nagambie, which served to divert water to the irrigation-districts on either side of the river.   Following the expansion of the system, the Waranga reservoir was constructed between 1902 and 1905 to a capacity of 201,300 acre-feet, and increased 14 years later to 333,400 acre-feet.

In response to the demand for further storage-capacity, the construction of the Eildon dam was undertaken in 1912, on the Sugarloaf site immediately below the junction of the Delatite and Goulburn rivers, about 150 miles above the Goulburn weir.   The rockfill dam, with a concrete corewall and spillway-section, over 3,000 feet in length and 140 feet in maximum height, was practically completed in 1927.

The Irrigable Area.

The Goulburn valley (*Fig. 1*) extends from the mountainous country in the eastern portion of the State, traversing the country north of the Great Dividing Range in a westerly direction for about half of its length, then northward and westward through the plains of the south-eastern extremity of the Murray basin ; it joins the Murray valley proper in the vicinity of Echuca.   The plains comprise

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[1] " Irrigation Works and Practice in Victoria," J. S. Dethridge, Trans. I.E. Aust., vol. 2 (1921), p. 93.

the rich alluvial deposits of the old Murravian Gulf, and that portion of them which lies in the State of Victoria embraces practically the whole of the irrigable area commanded by the Goulburn and complementary systems north of the Dividing Range, and includes districts

Fig. 1. THE GOULBURN IRRIGATION-SYSTEM.
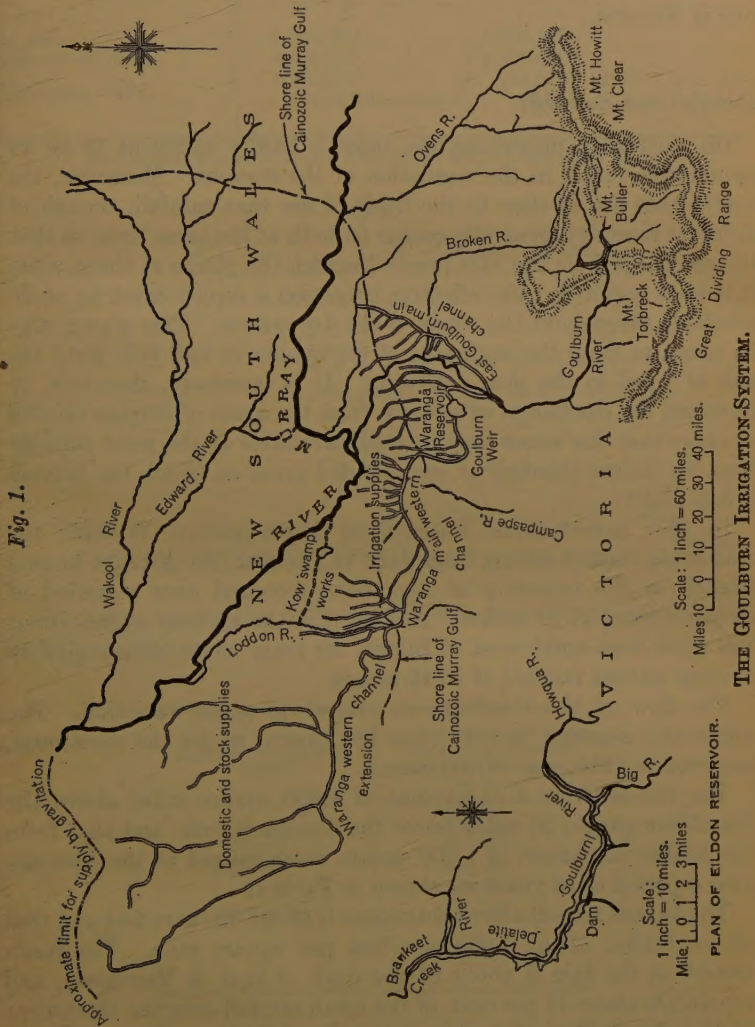
devoted to irrigation of a more intense nature, as well as those, further west, where the provision of water for stock and domestic use constitutes the main purpose of supply. The aggregate area of these lands is about 4,000,000 acres, but even with the most thorough

system of storage and regulation that could be proposed, the available supply of water can never be sufficient for the whole, an area of 2,000,000 acres being the maximum which could be profitably irrigated ; therefore, of the three governing factors—labour, land and water—necessary for irrigation-development, the latter is the limiting one in Victoria.

Rainfall and Run-Off.

On Australian mountains the snow is hardly sufficient to be an appreciable factor in the regulation of the streams.   Moreover, the catchments are so close to the fields of use that rainfall, though of different intensity, generally occurs on both at the same time, so that the discharge from the hills passes through the plains at times when it is least required.   For effective irrigation a supply must be made available throughout the whole of the dry season.   This season may be taken as from the beginning of September in one year until the end of April in the year following.   It is necessary, therefore, to impound the discharges of streams from the areas of surplus rainfall in order that the water may be made available at the most suitable time for its use throughout commanded areas on which the rainfall is deficient.[1]

In the mountainous districts in north-eastern Victoria the maximum rainfall occurs from June to October, the average annual rainfall at the township of Alexandra recorded over a period of 52 years being 27·21 inches.   At Boort, in the western plains, about 140 miles west-north-west of Eildon, 40 years' observations give an average annual rainfall of 15·43 inches.

The flow of the Goulburn is subject to great variation.   The minimum, recorded in 1915, was 180 cusecs, whilst the maximum, recorded in 1916, was 80,000 cusecs.

The run-off from a catchment of 4,000 square miles above the Murchison gauge, 20 miles below the Goulburn weir, and that from the Eildon catchment of 1,500 square miles gauged at the dam-site over a period of 18 years are shown in Table I.

The annual run-off above Murchison is about 600 acre-feet and that above Eildon about 1,000 acre-feet per square mile.   The mean run-off at the dam is about 66 per cent. of that at Murchison, and represents about 47 per cent. of the mean rainfall over the catchment of 40 inches.   Typical weekly discharges at Eildon weir are shown in *Fig. 2.*
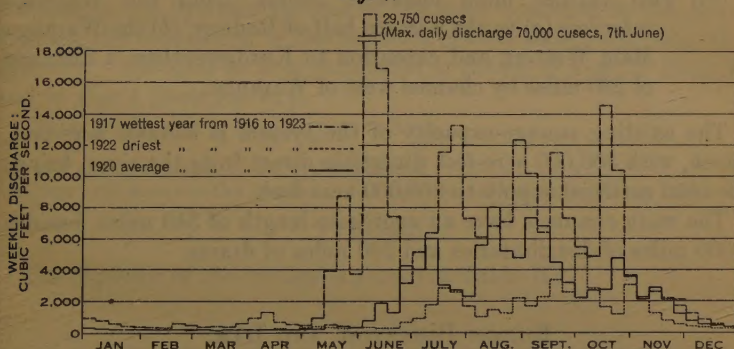
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[1] See J. S. Dethridge, " Irrigation Works and Practice in Victoria." Trans. I.E. Aust., vol. 2 (1921), p. 93.

TABLE I.

	Annual run-off : acre-feet.	
	Murchison.	Eildon.
Mean	2,353,000	1,554,800
Maximum (1916) . .	6,202,170	3,362,870
Minimum (1922) . .	{ 1,210,000 { + 493,000 diverted *	744,010 (1914 not included)

* Maximum diversion above Murchison 807,000 acre-feet in 1927.

Fig. 2.

The areas within the belts of different intensity of rainfall are shown in Table II.

TABLE II.

Rainfall per annum; inches.	Area of catchment above dam-site at Delatite-Goulburn junction : acres.
25–30	168,000
30–40	197,000
40–50	230,000
50–60	225,000
Over 60	160,000
	980,000

Engineering Works of the Goulburn System.

The chief works of the scheme (*Fig. 1*, p. 113) are as follows :—

(i) The Eildon reservoir, the main storage, which is of 306,000 acre-feet capacity (full-supply level being R.L. 823).

(ii) The Goulburn diversion-weir, situated 150 miles downstream from the dam, near Nagambie, which raises the level of the river 45 feet (to R.L. 405) and diverts the waters to :—

(iii) The Eastern Goulburn main channel (capacity 450 cusecs, 54 miles in length) supplying four districts surrounding the town of Shepparton.

(iv) The Goulburn-Waranga channel (capacity 1,717 cusecs, length 23½ miles), which supplies the eastern half of the Rodney main channels and fills Waranga basin.

(v) The Waranga basin, of storage-capacity 333,400 acre-feet (increased in 1919 from its original capacity of 201,000 acre-feet).

(vi) Two further main channels issuing from the Waranga reservoir (a) to the western half of Rodney, (b) the Waranga Main Western and extension to Kurdgweechee, a distance of 230 miles by channel west of Waranga.

The existing storage-capacity of the system is 660,000 acre-feet, which, with 300,000 acre-feet divertible direct from the river, brings the total artificial supply to 960,000 acre-feet.

The main channels have an aggregate length of 340 miles, besides 2,300 miles of distributaries and 500 miles of drains.

Eildon Reservoir and Dam.

The Catchment-Area.

The catchment-area of 1,500 square miles consists mostly of mountainous country heavily timbered below the snow-line. Even in the vicinity of the reservoir the topography is exceptionally steep, and within the watershed generally the streams have cut abrupt gorges between the thickly-timbered ridges.

The country is of a rocky nature throughout and the run-off is rapid, even over the northern and extreme north-western portions, where the wooded spurs give place to more open cleared country of a less precipitous character.

The catchment is divided into two main river-systems by the Enterprise spur, which extends for 45 miles in a westerly direction as part of the Howitt-Buller range (6,000 feet), an offshoot of the " Great Divide " which forms the eastern and southern watersheds. On the north of this formation is the Delatite river (*Fig. 1*, p. 113), with the Brankeet and Ford creeks as its more important tributaries, while to the south lie the Goulburn river and its confluent streams the Howqua, Jamieson and Big rivers and the Jerusalem creek. The Delatite and Goulburn rivers in the vicinity of the reservoir each

flow in a north-westerly direction on their respective sides of the Enterprise spur, the former flowing south and then south-east around its western slopes to join the Goulburn about ½ mile north-east of Sugarloaf, opposite the site of the dam.

At the full-supply level of R.L. 823 (123 feet above river-bed), the impounded water extends 11½ miles along the valleys of both the Goulburn and the Delatite rivers.

The Dam.

The dam, 3,259 feet in length, is situated just below the junction of these two rivers, and abuts at its eastern end on Mount Pinniger and at the western extremity on the Sugarloaf hill, from the side of which is quarried the bywash, the bed of the river Goulburn at this point forming the foundation of the concrete gravity-dam comprising the spillway. The dam as now completed is shown in Figs. 3, Plate 1, and in *Fig. 4* (facing p. 118).

The total length is made up as follows :—

Rockfill embankment . . . 2,525 feet
Spillway and power-outlet tower . 734 feet (actual crest of spillway 682 feet)

3,259 feet.

The capacity of the storage is shown in Table III.

TABLE III.

R.L.	Depth above river-bed : feet.	Area : acres.	Capacity : acre-feet.	Remarks.
706	3	—	—	Sill of main outlet.
713	10	—	175	—
743	40	820	4,050	—
753	50	1,400	14,600	—
754·5	51·5	1,425	16,550	Sill of power-outlet.
773	70	2,640	54,770	776·5 level at date of subsidence.
793	90	4,360	123,430	—
808	105	6,050	200,700	Level of gap in spillway 1929–33.
823	120	8,000	306,000	Full supply.
840	137	10,352	461,600	Crest of corewall.

The Main Embankment and Corewall.

Constructed of rockfill, the main bank supports a concrete corewall 6 feet thick at bedrock (R.L. 703) and 2 feet thick at crest level (R.L. 840), doubly reinforced by a grid of ½-inch diameter mild-steel rods spaced at 12-inch centres. Construction-joints were provided at distances of 50 feet throughout the length of the core. Originally

the highest point of the dam was at R.L. 840, 137 feet above bedrock. This was later increased by 16·7 feet on the downstream side.

The dam-site is of silurian age, the rocks of Mount Pinniger comprising shales of different degrees of hardness with an occasional band of slate, whilst at the Sugarloaf or spillway end they are generally of a more durable nature, being composed of somewhat similar though harder metamorphic rock as well as sandstones and slates. The country has been subjected to much folding, and fault-planes occur in the hills at each end of the dam ; a double fault-plane about 6 feet thick containing a mass of breccia abuts on the bywash, and has been revetted by a mass-concrete block keyed into the rock floor.

The embankment is composed of rock quarried from both Pinniger and Sugarloaf, broken to " one-man " size, hand-loaded, and tipped on the bank without being consolidated in any way. The stone from Sugarloaf resists weathering to a greater extent than the Pinniger mudstone, although the mixture forms a satisfactory fill.

The slopes were originally designed as 1 in 2, but having regard to the proposed second-stage development they were not trimmed, but were left in a series of berms to facilitate the tipping of the fill required in the proposed enlargement work when the full-supply level was to be raised to R.L. 875, the " mean slopes " on completion of the first stage being approximately 1 in 2 as designed (Figs. 5, Plate 1).

Clay Against the Corewall.

As a means of increasing the impermeability of the reinforced-concrete diaphragm, a mass of clay was deposited on its upstream side, ranging in thickness from 27 feet at chainage 850 from the eastern end to 37 feet at chainage 2,450. The upstream batter of this mass was originally designed as 6 to 1, and it was built up to R.L. 835, 5 feet below the crest level of the dam.

The clay puddle, excavated from the surface deposits adjacent to the dam, was raised simultaneously with the rockfill, being well pugged by wetting and trafficking with horse and dray, by which means it was carted to the site. The dimensions of the base of the dam and the clay core are shown in Table IV, p. 119.

By the end of 1925, the clay was completed to R.L. 835 between chainages 00 and 1,700. At this time the level of the clay to the west of this was left at R.L. 821, and, according to the progress plans, it appears to have been raised to finished level, R.L. 835, in July 1927. In the central portion of the dam, settlement was thus being made good for 3 years, till the end of 1928, and at about chainage 2,300 for about 1 year. *Figs. 6* and Table V (p. 119) give particulars of clay-levels and top-widths in 1927 and 1929.

Fig. 4.

GENERAL VIEW OF COMPLETED DAM, LOOKING WEST FROM MT. PINNIGER.

5121.

Fig. 7.

WESTERN END OF DAM, SHOWING ROCKFILL AND SURCHARGES, MAIN
AND POWER OUTLET-TOWERS, NEW SPILLWAY CONTROL-STRUCTURE,
. AND EASTERN END OF OLD SPILLWAY-CREST.

TABLE IV.

Chainage : feet.	Base width of dam : feet.		Base thickness of clay core.
	Upstream.	Downstream.	
850	125	135	27
1050	155	165	30
1250	167	175	30
1650	175	206	30
2050	190	196	30 (50 at R.L. 750)
2450	270	305	37

Figs. 6.

R.L. 840
R.L. 835
Full supply level
R.L. 823

As designed 1918 ....
Amendments 1925 ......

As constructed to 1927.

Scale : 1 inch = 40 feet.
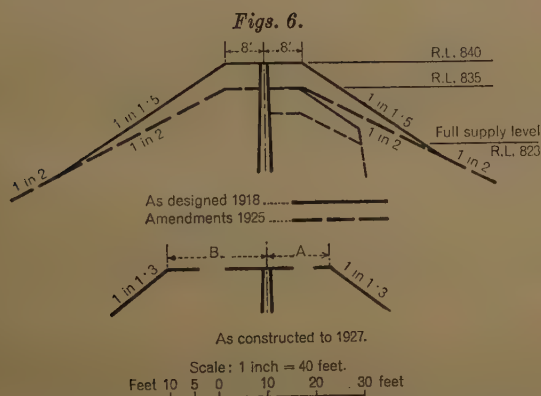Feet 10 5 0   10   20   30 feet

TABLE V.—DIMENSIONS OF TOP OF BANK IN 1927.
(Dimensions as increased in January, 1929, by additional rockfill are shown in brackets.)

Chainage : feet.	Upstream width A (*Figs. 6*): feet.	Downstream width B (*Figs. 6*): feet.
700	11 (27)	18
800	15 (28)	18
900	25	17
1,000	29	19
1,100	32	20
1,200	33	20
1,300	33	19
1,400	26	18
1,500	15 (30)	21
1,550	18 (28)	16
1,600	26 (28)	23
1,700	32	20
1,800	30	13
1,900	36	20
2,000	31	26
2,100	27	24
2,200	20	20
2,300	20	18
2,400	25	16

Foundation of Dam.

The western 1,000 feet of the dam, including the spillway section, are founded on bedrock, the remainder being founded on the natural surface. On the flats, the bedrock is from 22 to 24 feet below the surface, the overlying material being composed of clay super-imposed upon gravel wash with about 2 feet of sand immediately above the rock. In order to effect a junction between the corewall and the rock, an open cut extending from chainage 1,600 to 2,250 was excavated in this material, 24 to 28 feet wide at the top and 8 feet at bedrock level. Grout-holes 10 feet deep at 8-foot centres were drilled in the rock from chainage 1,600 to 2,600 and grouted with neat cement at 100 lb. per square inch. Where the depth exceeded about 23 feet a timbered trench was sunk to bedrock, its maximum depth being 77 feet at chainage 550. The gravel wash excavated from the open cut was deposited on the upstream side of the cut in a low spoil-bank about 100 feet wide at the base, whilst that from the deep trench on the slopes of Mount Pinniger was dumped from skips into heaps extending normally to the corewall, two on the upstream and two on the downstream side of the centre-line of the dam.

The Spillway.

The spillway was built as a curved concrete dam of gravity section 682 feet in length and 85 feet wide at the base (R.L. 700), with its crest at R.L. 823 (the original full-supply level). A protective apron at R.L. 712 about 7 feet thick extends for a maximum distance of 310 feet downstream of the toe. On the western or bywash side, the unlined rock rises to R.L. 808. The proposed raising of the full-supply level to R.L. 875 would have increased the storage-capacity from 306,000 acre-feet to 900,000 acre-feet. As a provision for bonding the concrete in the raising of the spillway, the downstream face was formed into fourteen steps 6 feet high and 4 feet wide, which acted as an effective energy-dissipator during operation at first-stage level. The apron at R.L. 712 extends 40 feet downstream from the lowest step. The spillway as now completed by the addition of gates (as described later in the Paper) is shown in Figs. 24, Plate 2, and in *Fig. 7* (facing p. 119).

Outlets.

There are two outlets for the control of discharge from the reservoir, the main outlet and the emergency or power outlet.

Main Outlet.—This consists of a reinforced-concrete outlet-tunnel or culvert surmounted by a circular control-tower rising through the upstream rockfill, into which are built the gate-valves, pipes and

operating machinery. It is shown as finally re-modelled in Figs. 21, Plate 2. Penetrating the centre-line of the dam at chainage 2,250, the culvert, originally constructed to by-pass the Goulburn river during the construction of the spillway, is founded on rock at R.L. 700 ; from the corewall it extends 250 feet upstream to the toe of the dam, and 410 feet downstream to the outlet into the stream-bed.

The control-tower is 68 feet upstream from the corewall. Set in concrete, which fills the downstream half of the tower below R.L. 774 and the outlet-tunnel from the east–west diameter of the tower to the corewall, are four 4-foot 6-inch cast-iron pipes comprising the outlet-system. Originally, water was admitted to these pipes through eight openings each 5 feet by 3 feet, set in two tiers of four, with their sills at R.L. 706 and 754·75. From the lower pipes in the tunnel-plug it passed through transition-castings at the centre-line of the dam into ovoid conduits of reinforced concrete 7 feet 6 inches by 4 feet 6 inches, extending 200 feet from the corewall into the tunnel. At a distance of 10 feet 6 inches from the valves these pipes were joined by down-pipes 4 feet 6 inches in diameter leading from the upper tiers. All valve-leaves were the same size, namely 5 feet 9 inches by 3 feet 7 inches, and were operated by hydraulic lifters. It had been anticipated that both tiers of valves could be operated simultaneously, but this was found to be impracticable (p. 158), although irrigation-requirements were satisfied by a careful manipulation of the gates during the construction period and up to 1929.

The Emergency or Power Outlet.—This, as finally modified, is shown in Figs. 23, Plate 2. An opening 19 feet by 16 feet in the eastern end of the spillway was provided to admit water to a 15,000-kVA. hydro-electric power-station situated immediately below the dam. From this opening three 10-foot by 4-foot outlets with sills at R.L. 754·50 lead through a transition casting 12 feet in diameter, and a tapered mild-steel bend 40 feet in length, to a 13-foot 6-inch mild-steel riveted pipe $\frac{3}{4}$ inch thick embedded in concrete. At the power-house end of the pipe a three-way riveted steel spherical manifold leads the water either to the turbines or to a Glenfield—Kennedy needle-valve (108-inch inlet, 90-inch outlet) which enables the outflow to be by-passed direct to the river. The three 10-foot by 4-foot outlets are controlled by gate valves enclosed in a rectangular tower, 50 feet by 19 feet 8 inches.

The opening and pipe-inlet were protected by a concrete-covered grid of four 10-inch by 6-inch by 40-lb. joists supported at mid-span by a 24-inch by 7$\frac{1}{2}$-inch by 90-lb. joist. Downstream of this a steel trash-rack was slung, made up of 2$\frac{1}{2}$-inch by $\frac{5}{8}$-inch flats spaced at 4$\frac{5}{8}$-inch centres and supported on a frame of steel joists, providing a maximum opening of 6 feet 4 inches by 4 inches which was later

reduced to 2 inches by $1\frac{11}{16}$ inch by superimposing a mesh of $\frac{5}{8}$-inch diameter welded mild-steel bars over the original framework. The resulting loss of head across the screen caused a great reduction in the maximum possible flow through the power-outlet, for, at the point of maximum efficiency of the turbines under the head corresponding to a full reservoir (from about R.L. 808 to R.L. 823) with the by-pass closed, any further opening to increase the flow caused the head to fall away, resulting in surging of the electrical load accompanied by considerable noise in the turbines, governors, and the hydraulic equipment generally.

Table VI shows the theoretical discharges through the upper gates and the turbines simultaneously.

TABLE VI.—THEORETICAL DISCHARGE THROUGH FOUR UPPER GATES, TWO TURBINES AND BY-PASS.

Axis of gates at R.L. 757.

Water-level: R.L.	Head H: feet.	$\sqrt{H}$.	Discharge : cusecs.			
			Four gates.	Two turbines.	By-pass.	Total.
763	6	2·45	1,019	1,390	1,230	3,539
773	16	4·00	1,664	—	1,640	—
783	26	5·10	2,122	1,470	1,640	5,232
793	36	6·00	2,496	1,560	1,810	5,866
803	46	6·78	2,820	1,660	1,970	6,450
813	56	7·48	3,112	1,760	2,130	7,002
823	66	8·12	3,378	1,800	2,270	7,448
833	76	8·72	3,628	1,800	2,370	7,790

Actually the greatest discharge obtainable through the power-outlet was about 2,000 cubic feet per second, while that through the gate valves was limited by the conditions arising in the conduits during operation (p. 158).

Drainage of Dam.

To provide for the disposal of leakage through the corewall and natural drainage under the downstream rockfill, a vitreous earthenware pipe 3 inches in diameter was laid along the corewall at the bottom of a French drain. This extended from chainage 650, where its invert-level was R.L. 735, to the inspection-shaft at chainage 1,550, which it entered at R.L. 725, 4 feet above the bedrock. The inspection-shaft was built on to the corewall, and gave access to its downstream face from the crest of the dam (R.L. 840) to bedrock-level. From the western side of this shaft the drain was continued in a westerly direction to the wall of the main outlet-culvert at

chainage 2,240, where it entered a junction-chamber (R.L. 717·50) whence it discharged either direct into the culvert or to the river through a drain embedded in the arched wall. At the inspection-shaft the drain from the eastern section could be blocked by closing a 3-inch sluice-valve, thus isolating the drainage from each portion of the bank for separate gauging if necessary.

Drainage through the concrete gravity spillway-section passed into the inspection-gallery, on each side of the floor of which an open drain was constructed, discharging to the bywash.

Periods of Full Reservoir.

Prior to April, 1929, due to the expansion of the Goulburn system during the construction period, it was necessary to store water for

Fig. 8.
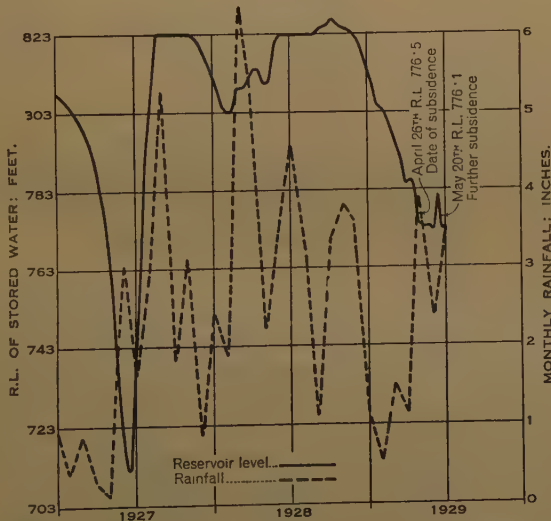
RESERVOIR-LEVELS AND RAINFALL AT EILDON RESERVOIR.

irrigation. The quantity of stored water increased from 54,000 acre-feet in 1925 to 211,000 acre-feet in 1926, the reservoir being filled to its full capacity, 306,000 acre-feet, for the first time in August, 1927 (*Fig. 8*), where it remained for about 2½ months before it was drawn down 20 feet to R.L. 803, over a period of about 10 weeks commencing in the middle of November, 1927. In the previous year, during the emptying of the basin, the draw-off had been much more rapid and prolonged, for, in the 6 months January to May, the

reduction of level was 106 feet in 24 weeks or approximately 4 feet 5 inches per week.

In February, 1928, the reservoir again began to fill, reaching full supply level in mid-June, where it remained until the end of August. The level then gradually rose above R.L. 823 until October 6th, 1928, when it was 4 feet 2 inches over the spillway. From the end of November of that year to April 26th, 1929, the average draw-off was 2 feet 4 inches per week, slightly faster than in 1927. The weekly draw-offs from January to May, 1929, are given in Table VII.

TABLE VII.—WEEKLY FALL OF STORAGE-LEVEL, JAN.—MAY, 1929.

Week ending	Fall : feet.	Week ending	Fall : feet.
Jan. 7 . . .	2·1	March 18 . .	2·70
„ 14 . . .	2·9	„ 25 . .	3·55
„ 21 . . .	3·0	April 1 . .	2·7
„ 28 . . .	1·6	„ 8 . .	Rise 0·6
Feb. 4 . . .	0·85	„ 15 . .	1·7
„ 11 . . .	1·95	„ 22 . .	4·4
„ 18 . . .	2·3	„ 29 . .	6·1
„ 25 . . .	1·65	May 6 . . .	0·05
March 4 . .	1·45	„ 13 . .	0·10
„ 11 . .	1·85	„ 20 . . .	0·30

THE SUBSIDENCE OF THE BANK.

On the 26th April, 1929, when the level of the reservoir had been drawn to R.L. 776·5, 46 feet 6 inches below its level of 5 months previously, a subsidence occurred of the rockfill on the upstream portion of the bank over the central 700 feet. A further lowering of the reservoir-level to R.L. 769·5 was followed by a rise early in May to R.L. 778. The level again fell to R.L. 776·1 on the 20th May, when a further subsidence took place extending the affected portion of the bank to 1,200 feet, between chainages 850 and 2,050. The draw-off from the reservoir was discontinued some days later, and the level maintained at about R.L. 776. As a result of this subsidence the corewall was exposed for a maximum depth of 26 feet from the crest (R.L. 840) at chainage 1,550, and the upstream rockfill at this chainage was moved bodily into the reservoir for a distance of about 55 feet. The bank continued to subside slowly for some weeks after the first extensive movement. Sections of the dam before and after the subsidence are shown in Figs. 5, Plate 1.

The greatest subsidence took place against the corewall, and the exposed face was heavily scored with striations due to the pressure of the stone sliding against the concrete. In the immediate vicinity

of chainages 1,450 and 1,600, these markings were vertical, and between this zone and the extremities of the affected length they were inclined to the vertical, the inclination increasing towards the ends. This scoring of the face of the corewall was distinct evidence of the direction of the rock-movement, and indicated in some measure the nature of the subsidence. A serious deflexion of the corewall was disclosed in both the central and western straight sections of the dam, which are respectively 800 feet and 400 feet in length, and the maximum displacement of the diaphragm amounted to 4 feet 8 inches in each instance. In addition, considerable cracking of the corewall took place.

An extensive examination by survey, boring and shaft-sinking was undertaken to determine the full effects of the subsidence, pending the investigation of the occurrence by an independent authority.

On the 24th May, 1929, the Governor in Council appointed the following Board of Inquiry to investigate and report upon the cause, extent and appropriate treatment of the subsidence :—

> Professor R. W. Chapman, C.M.G., M.A., B.C.E., Professor of Engineering, University of Adelaide (Chairman) ; Mr. H. H. Dare, M.E., M. Inst. C.E., Commissioner, Water Conservation and Irrigation Commission, New South Wales ; and Mr. E. G. Ritchie, M. Inst. C.E., Engineer of Water Supply, Melbourne and Metropolitan Board of Works, Victoria.

The full effect of the subsidence at chainage 1,550, the point of greatest settlement, will be observed from an examination of Figs. 9, Plate 1, which show the cross-section of the dam as determined by boring and survey prior to the 12th June, 1929. It will be seen that the clay, which, as originally constructed, stood against the corewall in a mass, the upstream slope of which was 6 to 1, had been compressed into a lenticular mass penetrating the rockfill to a distance of 80 feet from the initial position of the corewall.

The depth of rockfill over the clay was 35 feet, instead of the designed 5 feet, whilst the displacement of the upstream toe of the bank increased progressively towards the central chainage (1,550) practically uniformly and in direct proportion to the subsidence of the clay, the maximum displacement, as stated above, being 55 feet. Of this, the outermost 10 to 15 feet would probably represent loose rock fallen from the slope during movement.

The Board found that the subsidence was due to the pressure of the clay causing the rockfill, placed on its upstream side as a support for the mass, to slide on the seat of the bank, into the reservoir.

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This pressure, the horizontal component of which caused the out-ward movement of the rock, may be said to have been due partly to that derived from the inherent properties of the clay, and partly to the loading of the clay by the superimposed mass of rockfill.[1]

With regard to the pressure derived from the inherent properties of the clay, certain aspects of the behaviour of clay, which were ascertained as a result of investigations by Mr. A. L. Bell, M. Inst. C.E.,[2] appear to be directly applicable to the case of Eildon weir, where a moist clay is subject to considerable pressure. Experiments, based on Mr. Bell's work, were carried out by Professor Chapman at the University of Adelaide on samples of clay taken from the bores sunk through the rockfill during the above-mentioned survey.

Mr. Bell derived a formula for the determination of the horizontal

Fig. 10.
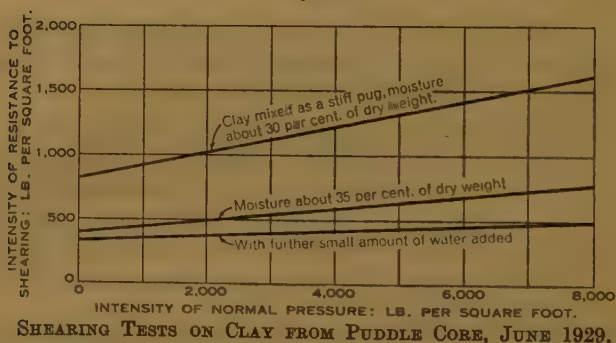
SHEARING TESTS ON CLAY FROM PUDDLE CORE, JUNE 1929.

component P lb. per square foot of the pressure due to a depth h feet of clay, namely :—

$$P = wh \tan^2 (\pi/4 - \alpha/2) - 2k \tan (\pi/4 - \alpha/2),$$

where w denotes the weight in lb. per cubic foot ($= 123$ for Eildon clay pug),

k denotes the resistance of clay to shear under no load in lb. per square foot, and

α denotes the inclination of the line of the graph shown in *Fig. 10*.

The increase in the horizontal pressure with increase in moisture-content of the samples of clay tested is shown in Table VIII.[3]

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[1] A copy of the Report of the Eildon Weir Inquiry Board (dated the 9th July, 1929) accompanies the manuscript of this Paper, and may be seen in the Library of The Institution.—SEC. INST. C.E.

[2] " The Lateral Pressure and Resistance of Clay, and the Supporting Power of Clay Foundations." Minutes of Proceedings Inst. C.E., vol. cxxix (1914–15, part I), p. 233.

[3] Report of the Eildon Weir Inquiry Board, pp. 18–19. Melbourne, 1929.

According to this theory,[1] the pressure exerted by the clay was equivalent to that of a fluid weighing about 100 lb. per cubic foot, which was resisted by the submerged rockfill and the hydrostatic pressure of the stored water.

It was held that, due to successive wettings, the moisture-content of the puddle wall increased, and that the great consequent increase in pressure was largely, if not wholly, the cause of the movement. That such an increase in moisture-content was possible in such a mass is not doubted, but clay taken from shafts which the Author caused to be sunk in the bank near the water-surface showed that there had not been a sufficient increase in the moisture-content of the mass to prevent its still possessing the qualities of good stiff pug, although the actual increase, if any, was not then determined.

Samples of the clay taken from the bores, after having been received in Adelaide in airtight containers, were "mixed and kneaded

TABLE VIII.

Sample.	k : lb. per square foot.	a	P (at 50 feet) : lb. per square foot.	P (at 100 feet) : lb. per square foot.	Weight (lb. per cubic foot) of equivalent fluid to produce	
					P at 50 feet.	P at 100 feet.
Stiff pug—						
30 per cent. moisture	820	5° 10′	3,500	8,500	70	85
35 per cent. moisture	396	2° 38′	4,700	10,160	94	101½
40 per cent. moisture	333	1° 05′	5,110	10,870	102	108

thoroughly with sufficient additional water to increase the total moisture content to 30 per cent. of the dry weight." The samples as received contained 23 per cent. of moisture and were too stiff for kneading for testing. A ball 3 inches in diameter formed from the pug and immersed in water collapsed into a cone-shaped mass after 24 hours, whilst cylinders 15 inches in length and 1–1¼ inch in diameter supported their own weight when suspended from one end. From those tests the clay was considered to be satisfactory for use in a puddle core. Further particulars of the tests made on the clay are given on pp. 186–190.

With regard to the pressure due to loading of the clay by the superimposed rockfill, the progress plans of the construction of the bank show that the clay had been brought to R.L. 835 as far west as chainage 1,750, and to R.L. 821 beyond that chainage, by the end of 1925 ; the rockfill throughout the length of the bank was first raised to R.L. 840 in 1927, the designed top width being 16 feet.

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[1] Report of the Eildon Weir Inquiry Board, pp. 18–19. Melbourne, 1929.

Probably as a provision for the subsequent dressing of the slopes, or the raising of the bank to the second stage, extra material was tipped at crest-level, and the top width was increased, ranging from 29 feet (at chainage 700) to 57 feet (at chainage 2,100) thus loading the clay to a greater extent than would have occurred in the original design. Between 1927 and the end of 1928, the upstream width at chainages 1,500 and 1,550 had been increased, by 15 feet and 20 feet, to 30 feet and 28 feet respectively. (See *Fig. 6* and Table V., p. 119.) Between October and November, 1928, Pinniger rock was tipped against the corewall to make good a small subsidence of the rock on the upstream side near the outlet-tower. Small quantities had been tipped along practically the whole length of the dam after 1927.

It would appear from progress records and ultimate boring that, between December, 1925 and December, 1928, settlement due to the sinking of the clay was made good by tipping rock to an ultimate depth of 35 feet over the clay, against the corewall. During this time the weight thus imposed upon the clay compressed it and caused it to displace the adjacent upstream rockfill until sufficient passive resistance was developed within the latter (without substantially changing its outward shape) to support the load transferred to it through the clay. This seems evident from the section at chainage 1,550 (see Fig. 9, Plate 1), where the clay appears to be forced into the upstream fill as a mass and not merely into the interstices of the fill, the individual rocks having " telescoped " to make room for it while developing the passive resistance to withstand the clay-pressure.

It will be noted from the corewall-deflexion graph (*Figs. 11*) that a downstream movement of the corewall had commenced in 1924, and it seems evident, in view of the mass of fill tipped against the corewall since then, that this movement continued up to the time of the subsidence, although there are no records to show that that was actually the case.

The gradually-increasing outward pressure of the clay was successfully resisted between 1924 and early in 1929 by the frictional grip between the upstream rockfill and the natural surface. During the submersion, however, this grip was lessened, and at the time of the subsidence, with the drawing-down of the reservoir to R.L. 776 and the consequent removal of much of the support of the stored water, the clay-pressure finally overcame the resistance to sliding on the seat, and the upstream rockfill began to move. It is not known to what extent the frictional resistance at the plane of contact between the rockfill and its foundation was lessened by the layer of clayey gravel that had been tipped as spoil from the cutoff trench along

the upstream surface in the vicinity of chainage 1,500 (p. 120), but it may be assumed that the continued submergence of this material largely contributed to the reduction of the frictional coefficient of the seat of the bank at the point where initial movement of the rockfill occurred.

As to the nature of the movement, reference has already been made

Figs. 11.

Chainage : feet	1,550	2,300	2,400	2,500
Height of crest (at R.L. 840) above natural .surface	97'	115'	no overburden	
" " " " " " " " rock	118"	128'	136'	140'

MOVEMENT OF DAM, AS SHOWN BY COREWALL-DEFLEXION SURVEYS.

to the markings on the face of the corewall. The vertical scratches near chainage 1,550 indicate a central downward movement, and those on either side the simultaneous longitudinal movement towards the centre. This would tend to demonstrate the occurrence of a plastic flow of the entire mass of clay under great pressure over the deepest portion of the bank, and bears out to some extent the statements of previous observers. For example, Mr. A. S. E. Ackermann,

during the years 1919–1924, conducted a series of experiments ; [1] among the properties attributed to clay by him are the following, which may be said to be directly applicable to the mass of puddle clay in the Eildon embankment :—

> (a) Clay has a definite pressure of fluidity, which, when developed and maintained, causes the clay to yield indefinitely as a dense viscous fluid, the vertical and lateral pressures then being equal. The pressure of fluidity varies with the percentage of water, and with the temperature.
>
> (b) The coefficient of friction with constant water-content varies inversely as the square root of the pressure.

Mr. Ackermann's tests showed values of the pressure of fluidity ranging from over 10·5 tons per square foot for a clay containing 17·5 per cent. of water to 1·0 ton per square foot for the same clay containing 26·5 per cent. of water, the tests being carried out at temperatures between 46·5° F. and 65° F.[2] The rapid falling-off of the coefficient of friction in a mass of clay with increase of pressure was confirmed by Mr. P. M. Crosthwaite.[3]

In tests of the load-carrying capacity of clay, it is the difference in pressure between the area under load and the adjacent clay that induces plastic flow, the static head of the clay not being great enough to prevent it. In the case of the embankment at Eildon, the addition of fill at the corewall possibly produced the differential pressure necessary for the outward movement.

The combined longitudinal and transverse movement of the clay was demonstrated during the early stages of the tipping of the new fill on the dam, when the light-gauge tracks running parallel to the corewall were, as a result of the former, subjected to considerable tension, often failing at the fish-plates ; whilst at the centre (chainage 1,550) the tracks were left suspended when the support was removed from underneath the sleepers by the outward movement of the rockfill. The rock mass at this period was in a state of continual movement, but no further rapid subsidence took place, although the corewall-deflexion increased during the restoration of the bank, and a considerable amount of material had to be placed, at the outset, to make good the settlement, before the new filling began to " take shape."

[1] " Experiments with Clay in its Relation to Piles." Transactions of the Society of Engineers, 1919, p. 37.

" The Physical Properties of Clay." Ibid., 1920, p. 196 ; 1921, p. 87 ; 1922, p. 151 ; 1923, p. 25.

[2] Ibid., 1920, p. 207.

[3] Ibid., 1919, p. 92.

The movement took place over that portion of the dam not supported directly on the bedrock; west of chainage 2,050, where the fill went down to bedrock, sufficient resistance against sliding on the seat of the bank existed, and the clay core over that section remained practically intact.

The pressure against the corewall from the upstream side was resisted by the rockfill downstream of the corewall, which, as in the case of the upstream fill, rested on the natural alluvial bed of the river valley, the foundation being saturated by seepage and surface-water. The rockfill weighed approximately 90 lb. per cubic foot, as compared with about 120 lb. for consolidated earth, and as no attempt at consolidation had been made by rolling or by the washing of fines into the interstices in the fill, some internal adjustment took place in developing the passive resistance necessary to withstand the corewall pressure; this resulted in a downstream deflexion of the diaphragm (*Figs. 11*, p. 129) without apparent sliding on the seat. Such movement could not take place without cracking of the corewall, particularly at the upper levels (see *Fig. 13*, p. 142) and partial failure along the base of the corewall at bedrock-level. A number of vertical cracks were apparent at the initial survey after the subsidence, the most serious being that at the junction of the core and " approach " walls. (*Fig. 14*, p. 143.)

Further evidence of the effect of the clay-pressure was observed in the deflexion of the main outlet-tower in an upstream direction, the shell of this structure at its junction with the outlet-culvert arch having cracked badly and endangered the safety of the outlet-works generally. (*Figs. 20*, p. 159.)

Decision to Restore the Dam.

In view of the extensive damage to the works, and the defects later disclosed, the Government was faced with two alternatives : (*a*) the breaching and abandonment of the dam, (*b*) its restoration to a state of safety and efficiency. The economic importance of the dam has already been emphasized, and the Inquiry Board came to the conclusion that the expenditure necessary to effect the restoration of the works " will not be more than is warranted to enable the previous large expenditure to be fruitful of results." The latter course was therefore decided upon by the Government of Victoria, and the extensive remedial work, together with improvements to the main and power outlets and the spillway, commenced in accordance with the general recommendations of the Board.

The damage resulting from the subsidence as discovered at the outset and during or subsequent to the investigations of the Board, the defects in the original design of the outlets and of the spillway,

and the manner of executing the remedial measures will be dealt with in this Paper under the following heads :—Rockfill, Corewall, Clay, Drainage, Main and Power Outlets, Spillway.

ROCKFILL.

An economical form of rockfill dam, wherein the rock is disposed to give the maximum stability for a given mass of material, would comply with the following requirements :—

(1) A foundation of sound clean rock free from overburden.

(2) A mass of durable rock, comprising the body of the dam, disposed with the largest rocks on the downstream side, and the materials as a whole equal in grading to quarry-run for that particular type of rock, the spaces between the rocks being filled with finer material and the whole forming a freely-draining mass with side-slopes somewhat less than the natural angle of repose.

(3) An upstream reinforced-concrete slab of the " articulated " type[1] supported directly on the fill and continued downward into the bedrock as a cut-off along the upstream toe, the depth of the cut-off being determined by the nature of the rock foundation.   The connexion between the slab and cut-off would be designed to allow of settlement of the fill by forming a " hinged " watertight joint with special provision for drainage.   With the upstream rockfill placed as a well-graded mass a uniform bearing for the slab could be prepared, and comparatively even settlement would be assured.

In a dam of this type the whole weight of the rockfill resists the water-pressure, the vertical component of which increases the total weight on the base and thus adds to the resistance to sliding on the seat.

With a central corewall much of this economical distribution of the rockfill is lost, as the chief function of the upstream portion is to support the diaphragm, and, with the provision of a puddle core supplementing the corewall as an extra aid to impermeability, the upstream rockfill is called upon to exercise the added function of supporting the clay-pressure. Large internal forces are thus developed within the dam, which can only be resisted by masses of fill greatly in excess of the quantity required for stability in the type with the upstream slab.

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[1] A laminated type of upstream slab has recently been employed on the San Gabriel No. 2 dam, California ; see *The Excavating Engineer*, vol. 27, part 5, May, 1933.

It is advantageous to place hand-packed rubble on either side of the wall to ensure the immediate development of the maximum passive resistance in the adjacent fill. In the Oued Kebir rockfill dam recently built in Tunisia [1], the extreme downstream portion of the dam was constructed in the form of a hand-packed mass of trapezoidal section, between which and the central corewall rockfill was tipped ; the active pressure of the latter reacted against the hand-packed section and opposed the combined pressures of the upstream rockfill and the stored water. The corewall side of the packed material was faced with concrete to provide a smooth surface down which the fill could slide freely in its wedge-like action against the corewall. In some recent American dams, a large-scale modification of the hand-packing method has been used, boulders up to 7 tons in weight being placed by cranes, with the object of interlocking them so as to prevent readjustment under pressure.

Consolidation of rockfill may also be attained by tipping quarry fines amongst the larger stones, or by washing-in sand during construction.

As already described, the Eildon dam, for the greater part of its length, was founded on clay, and the rockfill remained as deposited from side-tipping trucks. It therefore suffered from two important disadvantages—(a) the lack of consolidation and the resulting low value of passive resistance of the downstream portion, and (b) a low resistance to sliding on the seat of the bank. Under the disrupting influence of the clay-pressure (supplemented by the weight of the rockfill superimposed on the clay) these weaknesses were manifest in the deflexion of the corewall and the displacement of the upstream portion of the bank.

Remedial Measures Recommended.

After the subsidence, an expeditious means of increasing the passive resistance of the downstream fill and the resistance to sliding of the upstream mass was to add weight, in the form of rockfill, to the appropriate portions of the dam. The procedure recommended by the Board of Inquiry to effect this was as follows :—

> "It is recommended that as soon as possible efforts be concentrated on placing the filling required above that portion of the bank on the upstream side of the clay. This should be brought up to the level of the berm at R.L. 814, and the extent to which additional filling should be added above this level will

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[1] " L'Alimentation en Eau de Tunis " ; *Revue Industrielle*, vol. 61, no. 2066 September, 1931. (See also *Engineering News-Record*, 3 November, 1932.)

depend upon further experience as to the stability of the bank.
Filling above this level should be added in the form of raised
sections shown on the drawing, on the broad general principle
that weight above water level is of greater value than weight
added below. Care should be taken to place only the hardest
available rock on the slopes next to the water. When this
operation is well advanced the bank adjacent to the core wall
may be completed to the top width and slope as shown on the
drawing." (See Figs. 9, Plate 1.)

The bank at the western end could not be extended until the com-
pletion of the blister-tunnel protecting the inlet leading to the power-
house.

The placing of the additional rockfill on the dam was to be carried
out in four successive phases, as follows :—

(a) To oppose the movement of the clay and prevent its further
subsidence, rock was to be tipped along the upstream toe
of the bank, forming a berm at R.L. 814 for a minimum
distance of 130 feet upstream from the true centre-line of
the dam. The water-face was intended to be formed to
1 in 1·5, but actually the slope was left at 1 in 1·3 as
tipped. On completion of the power-outlet works, the
upstream fill was to be extended westwards.

(b) To support the corewall against the pressure of the down-
stream fill, rock was to be tipped against it in such
quantity as not to overload the clay and thus induce
further movement, but yet to give adequate stability
against overturning of the exposed corewall. At crest-
level (R.L. 840) the width of this prism was fixed as 5 feet
from the true centre-line, and the upstream slope was to
be 1 in 2 ; this also was modified as the work progressed,
and the upstream slope trimmed to 1 in 1·3.

Until the work had progressed thus far, it was deemed
advisable not to empty the reservoir, but to keep the level
as nearly as possible to R.L. 776, to which the basin had
been drawn down when the subsidence occurred.

(c) To increase the supporting power of the downstream rockfill,
it was necessary to add to its mass and thus help to
increase its frictional grip on the natural surface. Rock
was therefore to be tipped over the whole downstream
face, the crest being widened from 20 feet to 60 feet ;
a thickness of 3 feet beneath the face was hand-packed
to give a uniform finish to the slope, which was left at
1 in 1·3. (Further steps to increase the frictional

resistance of the bank on the seat by draining the under-
lying strata are referred to on pp. 148–157.)

(*d*) Surcharges of rockfill, both on the upstream side of the dam
between R.L. 814 and R.L. 828, and on the downstream
side between R.L. 840 and R.L. 857·6, were to be added to
increase the support for the clay and the pressure against
the corewall respectively. The former would tend to
counteract the effect of the immersion of the fill on the
water-face, whilst the latter would add to the passive
resistance of the mass behind the wall by its compacting
effect.

To carry out the minimum requirements as above set out, it was
estimated that at least 700,000 cubic yards of material would be
required, distributed in approximately equal quantities upstream
and downstream of the corewall. The problem of placing such a
quantity of material on the dam in the minimum possible time at
once became one of major proportions, for, besides the immediate
necessity of fixing the upstream toe, it was imperative that the dam
as a whole should be restored to a condition of safety in such time as
to enable it to cope with the normal high-reservoir period and
subsequent draw-off during the irrigation season.

It was evident that such a complete restoration was impossible
before the expiration of one cycle, and the precautionary measure
adopted of lowering the full-supply level 15 feet, by cutting down
150 feet of the spillway-crest to R.L. 808, limited the possible head
on the dam to a minimum in case a condition of comparative stability
had not been attained in the 3 or 4 months that would elapse before
the next high-river period.

Method of Placing Fill.

In placing rockfill to form the original bank, material was trans-
ported from quarries at both the Sugarloaf and Pinniger ends in
horse-drawn rakes of 1-cubic-yard side-tip trucks. That from the
Sugarloaf end was placed directly on the lower levels or hauled up
ramps to higher elevations; at the Pinniger end, rakes were gravi-
tated to the dam by means of brake-drums, the loaded trucks in
their descent raising the empty ones to the assembly-points near the
quarries.

At the commencement of the remedial operations, in view of the
urgency of the work, these methods were again resorted to. The
stone, after being barred down from the quarry-faces, was broken to
" one-man " size and hand-loaded.

Other methods of working were considered, including that of

transporting rock from Sugarloaf by means of a ropeway supported on movable towers at crest-level, but these proposals would have involved various difficulties, including the opening-up of an old quarry with a 300-foot face, whilst the installation of the plant would have delayed the commencement of the work. In one quarry a mechanical loading method was tried for a short period, a $\frac{3}{4}$-cubic-yard dragline being converted for use as a crane and employed to hoist loaded skips from ground-level and tip them directly into the trucks. It was soon found that, although a slight saving could be effected by the method, the total output necessary, considering the urgency of the operations, could only be attained by the use of a number of such units, the cost of which, even had they been immediately available, would have been prohibitive.

It was therefore decided to continue loading by hand, working the more accessible Pinniger quarries with as many faces as possible. In order to expedite delivery to the dam, the horses hauling the rakes were replaced by petrol tractor locomotives.

Quarrying and Tipping Operations.

The two Pinniger quarries were situated on the north-western and north-eastern flanks of the spur abutting on the east end of the dam.

Working three shifts in the quarries was in this instance unavoidable, and a powerful system of floodlights was installed. Night operations on the faces, which reached a height of over 100 feet, were confined to drilling, in readiness for blasting on the following day, all barring-down of loose material being completed in daylight if possible. The rock was quarried to conform to a slope of about 40–45 degrees to the horizontal.

Two, and later three, sections were opened up in each quarry, and rakes of up to nine trucks used. While horse traction was in use, the rakes were lowered to the upstream berm at R.L. 814 on the brake-drum, but, with the introduction of tractor locomotives, which could negotiate the grades, and had other advantages, this plant was dispensed with and the loaded rakes were cleared more rapidly.

Progress of Work.

The quantities of rockfill placed during successive periods are shown in *Figs. 12*.

At the beginning of May, 1929, the full extent and nature of the subsidence were not known; as the immediate necessity was to prevent the collapse of the unsupported portion of the corewall, a trench was excavated along its downstream side and tipping was

commenced along the upstream side from tracks laid on the embankment. The rockfill continued to settle at the rate of about 1 foot per day for some days after the subsidence.

Before rockfill could be tipped along the toe of the clay, from the upstream berm, a ramp and tracks had to be laid and material made available from the operation of supporting the corewall. This

Figs. 12.

EFFECT OF ROCKFILL PLACED ON MOVEMENT OF DAM.

ROCKFILL SETTLEMENT AND COREWALL DEFLEXION.

commenced in August after 65,700 cubic yards of fill had been placed against the corewall, where during the next 3 months (up to the 1st November) a further 20,000 cubic yards were tipped, while 97,500 cubic yards were disposed along the upstream toe and 8,800 cubic yards on the downstream side.

During the period the old fill at chainage 1550 settled at a gradually decreasing rate to approximately R.L. 770.

For 12 months from November, 1929, only small quantities were tipped at the corewall, the majority going on to the downstream bank and the upstream berm approximately in the proportion of 5 to 3.

An interesting feature of the loading of the original fill on the downstream side was the sudden increase in the corewall-deflexion, that at chainage 1,550 being shown in *Figs. 12*. This was, in all probability, due partly to the " pyramidal " settlement of the old fill away from the diaphragm and partly to the compression of the supporting mass beneath it, the corewall having a considerable lean downstream.

A condition of comparative stability was reached in November, 1930, but with the resumption of tipping on the downstream side in November, 1931, the corewall-deflexion again reflected the movement in the bank. A period of consolidation then commenced, during which further movement was practically negligible. Observations of deflexions up to 1936 are given in Table IX, p. 140.

In widening the downstream portion, it was necessary to tip the rock from the crest down slopes lying at the angle of repose, upon which berms had been left after completion of the first stage. At the eastern end, to reduce this fall, the line was supported on one of these berms at R.L. 780 running out to chainage 1,050. The difficulty of holding the rock thus tipped on the slope was overcome by erecting fences of light rails about 4 feet high at intervals below the tracks, against which the falling material accumulated. The proportion of the fill which found its way to the bottom was small, and this surplus was later levelled on the natural surface in a series of terraces which gave added support to the toe. The entire downstream slope was finished by hand-packing a thickness of 3 feet.

Tipping at the western end was deferred until the completion of the protective blister-tunnel over the power-outlet in July, 1930, when the bank was extended along the approach-wall to chainage 2,515, at R.L. 840, the toe reaching to within 6 feet of the entrance to the blister-tunnel.

The rockfill was practically completed, except for small quantities placed in the western end and downstream surcharge, in 1932. The total quantity of rockfill in the dam as now constructed (Figs. 3, Plate 1) is 2,000,000 cubic yards.

COREWALL.

The thickness of central corewalls for rockfill dams has been the subject of mathematical treatment, but the results appear to vary considerably ; one writer states that a diaphragm of 2 feet in thickness from top to bottom has formed a satisfactory " stop for percola-

tion," whilst another in the case of a proposed dam to retain 96 feet of water gives a thickness ranging from 6 to 18 feet. It would thus appear that the design of the diaphragm is largely a matter of judgement and experience.

The septum of comparatively thin section provides the necessary flexibility in a rockfill dam, and with adequate reinforcement the occurrence of any crack of major dimensions is precluded. Provided that the dam stands on a suitable foundation and is properly drained, any leakage through the core will pass downstream without damage to the structure. The corewall of the Eildon dam is of this type, being 6 feet wide at natural surface and 2 feet at the top (see p. 117). Flexibility has been aimed at, a double grid of reinforcement being relied upon to bind the mass and to induce a series of minor cracks at points of maximum deflexion. The design was successful in that, despite the severe deflexion to which the wall was subjected at the time of the subsidence, it still remained practically impermeable.

Perhaps the most noteworthy feature in the behaviour of the corewall after the subsidence was that the exposed portion did not collapse upstream under the influence of the pressure of the downstream rockfill. The thickness of the wall at 26 feet below the crest was only 3 feet, but the pressure due to that height of rockfill was resisted by the doubly-reinforced concrete wall without any apparent upstream movement ; in fact, the downstream deflexion continued to increase. This would bear out the belief that, over a period of years, the rock particles of the downstream fill were being compressed and their frictional grip one with the other increased until the active pressure in an upstream direction was greatly reduced, possibly even below the " at rest " value originally existing. The case in point may be a verification on a large scale of the experimental results obtained by Mr. R. Stroyer, M. Inst. C.E., in his determinations of earth-pressure on flexible walls.[1]

The behaviour of the corewall both as to impermeability and deflexion was the subject of constant observation. Regular gaugings of the flow of water through the leaks and determinations of the corewall-deflexions were carried out throughout the duration of the remedial operations.

Deflexion.

In determining the deflexion of the corewall, permanent sighting points were established on both Sugarloaf and Pinniger, in continuation of the true centre-line, and the deflexions of the top of the core-

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[1] See Journal Inst. C.E., No. 1, Nov. 1935.

wall at 50-foot intervals were observed. A remarkably consistent series of readings, extending over more than 5 years, was obtained, observations being made daily for the greater part of the time, and, when movement became negligible, twice a week.

The deflexions of the wall thus recorded were of great interest. After periods of comparative rest it would resume its downstream movement for no apparent reason, and at times the differences indi-

TABLE IX.

Date.	Deflexions in feet at chainage :— (Downstream except where shown " U.S.")				
	950.	1150 (curve).	1550.	2050 (curve).	2300.
10 May, 1929.	1·06	0·13	4·52	0·97 U.S.	4·78
25 June, 1930.	1·41	0·18	4·90	1·49 U.S.	6·63
24 June, 1931.	1·70	0·02	5·03	1·50 U.S.	6·88
24 June, 1932.	1·76	0·07	5·12	1·46 U.S.	7·06
27 June, 1933.	1·74	0·10	5·21	1·51 U.S.	7·23
26 June, 1934.	1·74	0·10	5·29	1·54 U.S.	7·29
18 June, 1935.	1·81	0·12	5·40	1·50 U.S.	7·40
16 June, 1936.	1·82	0·08	5·41	1·54 U.S.	7·43

cated temporary minor upstream displacements. (See *Figs. 11* and *12*, pp. 129 and 137.) It was endeavoured to find some relationship between these variations and the factor or factors affecting them, such as reservoir-level, variations in pressure on the diaphragm due to unequal disposal of the rockfill, or the vibration of the bank resulting from the passage of the tractor-drawn rakes, but nothing (except perhaps the movement due to settlement of the downstream fill, referred to on p. 138) was of sufficiently definite character to be plotted in the form of a graph. (See *Fig. 12*.) During the period June 1929—May 1936, increases in the total deflexions were as shown in Table IX, the upstream movements, wherever recorded, being ultimately " cancelled " by the general downstream trend (see also *Figs. 11*, p. 129). There was comparatively little movement at the curves, the radii of which were each 700 feet.

Cracks.

The cracking which occurred in the corewall took the form of vertical, diagonal and horizontal fractures disposed throughout the diaphragm as follows :—

    (a) The vertical cracks occurred mostly above R.L. 823 throughout practically the whole length from chainage 950 to 2,150. At the western end (chainage 2,060 to 2,125) they

extended downward to R.L. 808 (*Figs. 13*, p. 142) and assumed a diagonal trend from east to west. The worst crack occurred at the junction of the corewall and approach-wall (*Figs. 14*, p. 143) at chainage 2,427, where an opening 1 inch in width occurred in the upstream face, accompanied, on the downstream side, by a compression failure with the usual flaking of concrete exposing the reinforcement in parts. It extended from the crest to R.L. 756. This was a typical failure in cross bending, the corewall being held in place solely by the $\frac{1}{2}$-inch-diameter reinforcement. The majority of the smaller cracks were no doubt caused in the first instance by thermal contraction and expansion (temperatures at Eildon ranging from freezing point to 110° F. in the shade), and were extended by the bending of the corewall.

(b) Many of the expansion-joints, left in the corewall at intervals of 50 feet, had opened up, and in some instances the concrete key, poured finally to close the joint, had cracked through. This condition occurred mostly west of chainage 1900, where all the joints had so failed, the cracks extending to R.L. 790 at 1,900 and shortening progressively to R.L. 817 at 2,150. East of this the extremities of these joint-cracks were found at levels between R.L. 777 and R.L. 801.

Except for that at chainage 2,427 feet, the vertical or diagonal cracks were of comparatively minor significance.

(c) Along the base of the corewall between chainage 1,530 and 2,200, a horizontal failure had occurred between R.L. 724 and 725, from 2 feet 6 inches to 5 feet above the surface of the bedrock and between 8 feet 6 inches and 11 feet above the bottom of the corewall bonding trench. Although it was not disclosed until the driving of the drainage-tunnel along the base of the diaphragm, calculation showed during the early investigation that such a crack was certain to have been caused by the known deflexion. On the downstream face it was a typical compression failure, flaking of concrete and exposure of the reinforcement having occurred, and it can be assumed that a corresponding tension failure with opening-up of the crack took place on the upstream side (*Figs. 13.*). This crack reappeared at the inspection-shafts over the outlet-tunnel at chainages 2,242 and 2,248 at approximately the same level, and probably ended

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Figs. 13.

PART UPSTREAM ELEVATION OF TOP OF COREWALL SHOWING JOINTS, CRACKS, ROCKFILL AND CLAY.

Note.—The broken lines with arrow-points indicate the direction of striations on the wall. Contraction-joints are at intervals of 50 feet. Where key intact, joint filled with non-porite.

PART DOWNSTREAM ELEVATION OF BOTTOM OF WALL IN TUNNEL.

PART DOWNSTREAM ELEVATION OF BOTTOM OF WALL IN TUNNEL.

TYPICAL CROSS SECTIONS OF FOOT OF WALL

TYPICAL CONTRACTION-JOINT IN COREWALL.

CRACKS IN COREWALL AFTER SUBSIDENCE.

near the thicker portion of the approach-wall some distance short of chainage 2,427.

Immediately east of the base of the shaft at chainage 1,530 a fractured zone, at about R.L. 724, was uncovered while driving the drainage-tunnel. Water found its way through this crack either as seepage or as jets spouting into the tunnel. At chainages 1,530 and 2,242–2,248, jets yielded a maximum of 24 gallons per minute and 12 gallons per minute respectively. The

Figs. 14.

CROSS SECTION AT 2,427 FT.
Scale: 1 inch = 160 feet.

SECTION AT FACE OF COREWALL.

PLAN OF SHAFT AT 2,427 FT
Scale: 1 inch = 12 feet.

PLAN SHOWING TOE OF CLAY AND PROBABLE LEAKAGE-PATH.

JUNCTION OF COREWALL AND APPROACH-WALL.
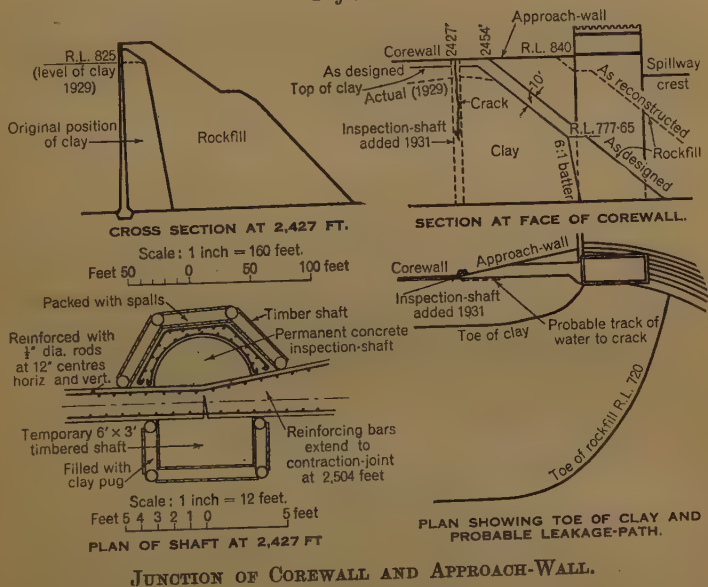
maximum drainage from all sources in the corewall was 48 gallons per minute.

These jets were thought to be fed by springs in the gravel wash rather than by water finding access direct from the reservoir, as the crack was below the open cut made prior to sinking the bonding trench, and therefore below the clay protecting the corewall. At chainage 2,242 to 2,248, however, the clay was directly opposed to the crack, and the water passing through the corewall at this break carried suspended clayey matter, having possibly found its way through shear cracks in the clay (*Fig. 15*, p. 144).

Treatment of Cracks.

All vertical cracks were followed down to their full extent by sinking timbered shafts 5 feet by 3 feet (over timbers) through the rockfill. Dovetailed chases about 2 inches wide, and the same depth, were cut along the cracks and then sealed in the following manner. Firstly, a composition of mortar mixed with asbestos fibre, previously soaked in water, in the proportions of

Fig. 15.
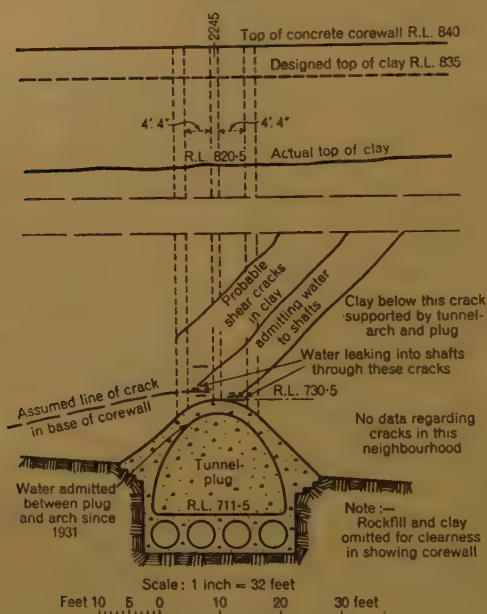
INSPECTION-SHAFTS AT CHAINAGES 2242, 2245, AND 2248, LOOKING DOWNSTREAM.

1 asbestos : 8 cement : 16 sand, was plastered on the sides and bottom of the chase. A heart of mortar composed of 3 parts of sand or toppings to 1 of cement was then tamped into the opening, leaving about $\frac{3}{8}$ inch of the chase to be finished off with the asbestos mixture. Shrinkage of this filling was negligible, and in the absence of further movement it proved an effective seal. The repaired cracks were then protected by a layer of clay concrete, consisting of 9 per cent. sand, 36 per cent. clay, and 55 per cent. quarry fines, mixed in a pugmill. When packed with wooden rammers in 6-inch layers, it adhered closely to the wall and was practically watertight. When placing the clay concrete, the shaft setts were lined with vertical

sheeting of either hardwood or red gum; this provided a comparatively smooth surface past which the material could settle if necessary.

The corewall-joints that had failed were chased and plastered as above and coated with a bituminous paint before filling the shaft with clay concrete. In the worst instance of a joint failure, which occurred at chainage 1,950, the upstream side of the crack was cut to form a dovetailed chase and filled with the cement–asbestos mixture, whilst the downstream side, where a second shaft had been sunk for the purpose, was caulked with oakum soaked in cement grout. Bitumen heated to the necessary fluid consistency was then poured in through short pipes left in the upstream filling at intervals of about 10 feet, to seal the joint. The usual clay-concrete filling was then tamped into the upstream shaft, and an inspection-well constructed on the downstream side as part of the drainage-system.

At chainage 2,427, two shafts were sunk, one on either side of the junction of the core- and approach-walls (*Figs. 14*, p. 143); they were made large enough to allow ample cover over the crack on the water face and sufficient clearance for a permanent concrete-lined inspection-shaft (subsequently incorporated in the drainage-system) on the downstream side.

On the upstream side the crack was 1 inch wide at the crest of the dam, and $\frac{1}{4}$ inch wide at R.L. 778, numerous branch cracks having formed simultaneously; yarn was employed to caulk the joint as a preliminary step. The flaked concrete was removed and chases cut along the smaller cracks where it was not possible to caulk. Cement mortar was tamped into the crack against the yarn backing and finished off with cement asbestos composition, as described above. On the downstream side no attempt was made to seal the break, but the damaged concrete, which as a result of subsequent movement may have become dislodged, was removed, in some instances further exposing the reinforcement.

Most of the shaft-sinking was completed in October, 1929, and the sealing to the old full-supply level, R.L. 823, by November of that year. In December the clay concrete was brought up to the level of the rockfill (R.L. 830), and by June, 1930, the filling had been completed to R.L. 836, above which level rockfill was tipped in, completing the upstream bank to the new section.

The shaft on the downstream side of the junction at chainage 2,427 was left as originally sunk (*Figs. 14*, p. 143) until August, 1931, when a commencement was made on lining it with reinforced concrete; the lining was completed in February, 1932. In designing the lining, no support was counted upon from the frictional grip on the corewall, and the " wings " were designed as cantilevers to

resist the rock-pressure. The weight of the lining was taken on a series of four steps between R.Ls. 716 and 724 (*Figs. 16*), and, to allow of free settlement of the fill past the concrete, the shaft timbers were lagged on the inside and the annular space packed with spalls. Six 6-inch diameter drainage-holes were left in the lining at R.L. 727·5, while at floor-level (R.L. 725·9) a similar drain was left to keep the shaft clear of ordinary seepage. The former would provide for any flow due to a fairly large possible future extension of the crack, and would prevent the shaft from filling, thus facilitating inspection.

Subsequent Behaviour of Crack at Chainage 2,427.

During October, 1934, a substantial increase in the water entering the shaft was noticed. At about R.L. 765, where previously only a dampness due to seepage had been noticed, water was jetting through the crack. This was kept under observation for some time, and it was observed that the jet always appeared with the surface of the reservoir water between R.Ls. 821 and 822. During the low-water period of 1935, the upstream shaft was opened up and the corewall at R.L. 821 examined. It was found that the crack had opened a further $\frac{1}{8}$ inch, and water, passing between the face of the wall and the shaft-timbers (between the inside and outside linings of which pug had been tamped during the placing of the clay concrete) had found its way through the reopened crack, down to R.L. 765, and thence into the inspection-shaft. The crack was treated as originally and the shaft again sealed.

Final Condition of the Corewall.

It is quite probable that throughout the length of the corewall during its continued deflexion, local adjustments resulting in the formation of fresh minor cracks have taken place, which are reflected in the increase at chainage 2,427. The question arises as to how far this adjustment will proceed before ceasing, and whether the junction-reinforcement will hold until that state is reached. The total yearly increases in the downstream deflexions at three points during each year since the subsidence are shown in Table X, p. 148 ; they indicate a gradual stabilization of the dam.

From those data it can be expected that no further serious cracking of the diaphragm will take place before a condition of practically complete settlement has been attained, and that its flexibility has enabled it to withstand successfully the severe treatment to which it was subjected during the subsidence.

The staunching of the horizontal crack along the base of the core-

Figs. 16.

REINFORCEMENT OF WALLS OF SHAFTS
AT 1,050 FT. AND 1,950 FT.

Position	Horiz. steel	Vertical steel
R.L. 720–740	$\frac{3}{4}''$ dia. at 5" c. to c.	$\frac{5}{8}''$ dia. at 12" c. to c.
740–760	$\frac{3}{4}''$ " at 6" c. to c.	$\frac{5}{8}''$ " at 12" c. to c.
760–780	$\frac{3}{4}''$ " at 5½" c. to c.	$\frac{1}{2}''$ " at 12" c. to c.
780–800	$\frac{5}{8}''$ " at 4½" c. to c.	$\frac{1}{2}''$ " at 12" c. to c.
800–820	$\frac{5}{8}''$ " at 6" c. to c.	$\frac{1}{2}''$ " at 15" c. to c.
820–840	$\frac{1}{2}''$ " at 9" c. to c.	$\frac{1}{2}''$ " at 18" c. to c.

Scale: 1 inch = 8 feet.

Inches 12 6 0 1 2 3 4 5   10 feet
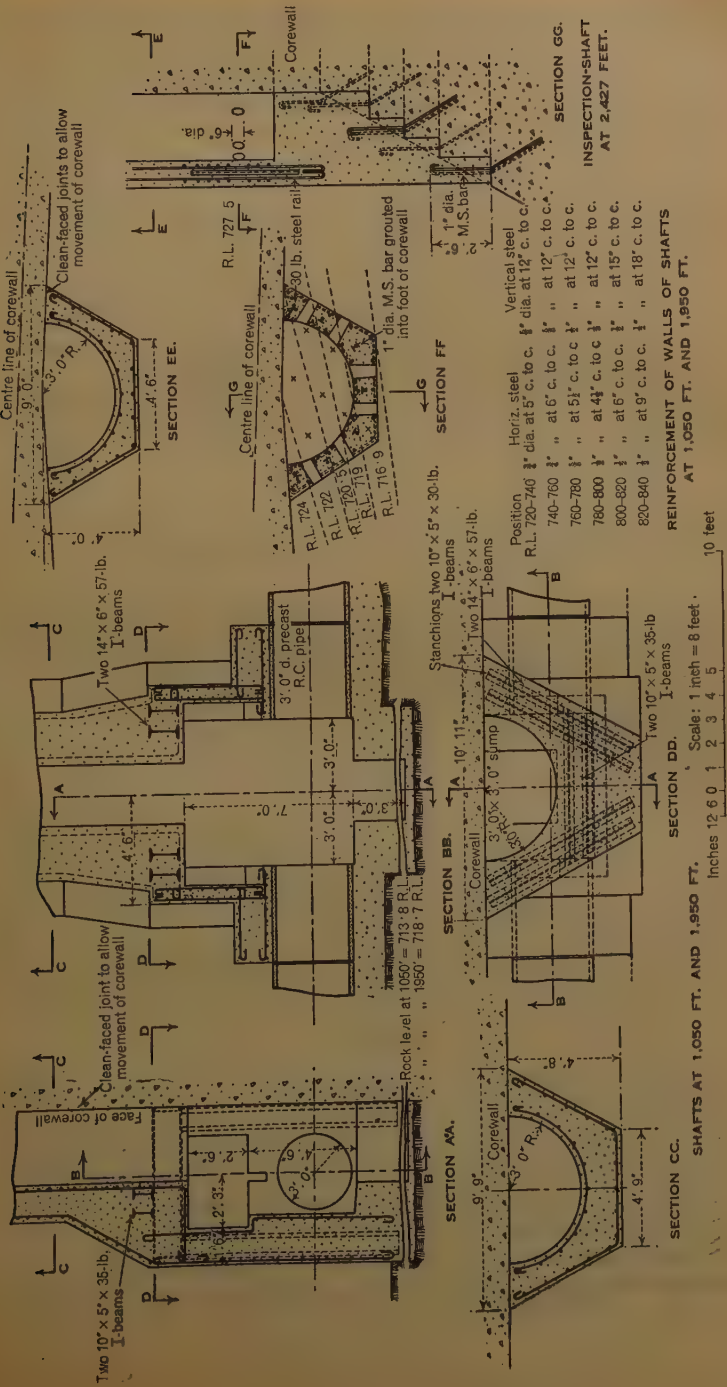
MAIN DRAINAGE-SYSTEM : COREWALL-SHAFTS.

wall was left until the concreting of the 3-foot-diameter corewall-drain (Figs. 17, Plate 1), when it was covered by the concrete in which the pipes were embedded.   This was considered a sufficient seal, and provision was made to admit any leakage passing the concrete into the drain through openings between pipe-lengths.

TABLE X.

Period.	Deflexion (feet) at chainage		
	1550.	1600.	2300.
Last 7 months, 1929	— 0·11	— 0·22	+ 0·90
Year 1930	+ 0·65	+ 0·65	+ 1·17
Year 1931	+ 0·01	+ 0·03	+ 0·11
Year 1932	+ 0·15	+ 0·14	+ 0·22
Year 1933	+ 0·04	+ 0·04	+ 0·08
Year 1934	+ 0·10	+ 0·11	+ 0·11
Year 1935	+ 0·08	+ 0·07	+ 0·06
First 6 months, 1936.	— 0·03	0·00	0·00

" — " denotes upstream, " + " downstream deflexion.

The methods adopted in sealing the cracks have proved quite satisfactory and, apart from any further minor breaks which may have been caused during movement since their staunching in 1929–1930, through which (if any) it has not been possible to detect any additional leakage, it may be said that the original condition of impermeability of the corewall has been successfully restored.

The total leakage through the diaphragm is subject to seasonal variation, as will be seen from the gauging graph (*Fig. 19*, p. 157). During a recent rise in the reservoir, July–August, 1936, from R.L. 763·5 to R.L. 821·95 the gaugings increased from 17·9 gallons per minute to 36·3 gallons per minute (including a certain amount of ground-water) which, for the total length of the corewall served by the drain (about 1,700 feet), represents a remarkably small leakage.

DRAINAGE.

In a rockfill dam with a central corewall, reliance is placed wholly upon the downstream portion of the bank to support the wall against the overturning effect of the combined pressures of the upstream rockfill and the stored water, and in order that the mass may be capable of developing its full reaction to these pressures any tendency of the downstream fill to slide on the supporting surface must be completely eliminated. Where the dam is founded—as in the case of the western 300 feet of the bank in question—on rock of an uneven

surface occurring in nearly vertical strata, the resistance to sliding is immediately developed, and is unimpaired by any subsequent wetting. The case is, however, quite different where the foundation consists of strata of gravel wash and clay superimposed upon the bedrock.

The greater portion of the Eildon dam rests on a foundation of this nature, the surface layers in direct contact with the rockfill consisting of loamy clay, which would possess little resistance to sliding after saturation with water, and the stability of the bank in general would thus be threatened if such a condition of saturation were allowed to persist. The intention, in the original design, was to convey the seepage through the corewall, and any water from other sources, to the outlet-culvert by means of a French drain incorporating a 3-inch earthenware pipe along the back of the wall. No other means was provided of preventing saturation of the foundation of the bank.

The nature of this foundation was made clear by the results of bores sunk near the downstream toe at the central part of the dam in June, 1929. These bores showed depths up to about 18 feet of clay of various consistencies, mostly wet. They did not disclose the sand stratum up to 4 feet in thickness which occurs immediately above the bedrock, the level of which, along the greater portion of the drainage-system, ranges from R.L. 720 to R.L. 724. Water rose in the boreholes to within a few inches of the natural surface of the ground.

At the time of the subsidence the ground near the toe of the dam from about chainage 1,200 to 2,000 was quite soft and wet. Spring waters were running over the surface in parts, and particularly in an area centred at about 300 feet downstream of chainage 1,200, where a saturated condition had been experienced for some time previously. The French drain was found to be blocked, and the water had dammed up to surface-level against the corewall, as disclosed by soundings taken in the 1,530-foot shaft. This accumulation of water resulted from surface runoff, leakage through the corewall, percolation through the gravel wash, and from water under pressure spouting through fissures in the bedrock—as was apparent on uncovering the rock surface during tunnelling.

Whilst no apparent movement had taken place in the downstream fill up to the time of the investigations into the cause of the subsidence, the condition of the ground underlying the bank gave cause for anxiety. The Board of Inquiry therefore recommended the installation of a corewall-drain (Figs. 17, Plate 1) driven along the back of the corewall from chainage 550 to 2,250 on the approximate level of the French drain. As this drain would take some time to complete, a more expeditious means of obtaining at least partial

improvement was also to be employed ; an agricultural drain was to be placed at a depth of about 4 feet along the ultimate downstream toe of the dam.   It was anticipated that this would drain the natural surface for a distance of 80 to 100 feet on either side of the line.   A third system was to be laid down to prevent surface-water from passing in under the dam.   This consisted of surface drains lined with rubble masonry laid along the hillside near the east end of the dam to convey such water into the depressions leading to the river.

The Corewall-Drain and Outlet.

This system as constructed (Figs. 17, Plate 1) consists of a 3-foot diameter drain running along the back of the corewall from chainage 1,040 to 2,200, falling from each extremity on a grade of 0·1 foot per 100 feet (except between chainages 1,530 and 1,200 where the grade was 1·1 foot per 100 feet) to a junction-chamber constructed below the inspection-shaft at chainage 1,530 with invert-level at R.L. 721.   The water collected is conveyed through a similar drain 250 feet in length lying at right angles to the corewall on a grade of 0·1 foot per 100 feet to a shaft giving access to the system from the natural surface (R.L. 743) at a point 80 feet from the toe of the dam.   In the south wall of this shaft is the entrance to the 2-foot-diameter outfall to the river.   Provision was made in the shaft for gauging the drainage-flow from the dam.   This 250-foot length of pipe effectively drains the gravel wash underlying the clay, and assists in drying-out the foundation for a distance of, probably, 300 feet on either side of the line of the pipe.

Between chainages 1,100 and 1,508, the 3-foot-diameter drain was driven under the natural surface at an average depth of 20 feet, and in order to lead water from the rockfill adjacent to the corewall into the system, five pipes or " risers " 12 inches in diameter, spaced at intervals of 100 feet, were constructed, passing through the clay and gravel to tap the French drain.   From chainage 1,530 westward, the corewall-drain traverses the rockfill immediately above the bed-rock, which extended half-way across the drive.   The 3-foot pipe throughout its length is thus in direct connexion with the rockfill ; it follows the horizontal crack at the base of the corewall, and the water from both these sources flows directly into the drainage-system.

The corewall-drain comprises a doubly-reinforced concrete pipe embedded for a depth of 2 feet in mass concrete founded on bedrock and extending across the drive from the timbering to the corewall. The pipes were specially made to the Commission's specification with sufficient strength to support a depth of 130 feet of rockfill ; they were 4 feet in length and placed with a gap of ¾ inch between pipes for the

entrance of drainage-water. Pressure-relief drains $2\frac{1}{2}$ inches in diameter at 4-foot 6-inch centres communicating with bedrock were cast in the concrete embedment, to convey spring water into the pipe from the foundations. The pipes were covered with a 12-inch layer of 4-inch crushed rock, and spalls up to 70 lb. in weight, or " one-man stone," were hand-packed between this and the tunnel-roof. With the ultimate rotting-out of the timbering, the weight of the superimposed rockfill would thus be taken evenly by the pipe.

At the junction-chamber at chainage 1,530, connecting the eastern and western corewall sections and the 250-foot length of drain leading to the main access-shaft and outlet, heavy reinforced construction was necessary where the drainage-tunnel skirted the base of the inspection-shaft (*Figs. 18*, p. 152); the overlying gravel and rock-fill were here supported on a roof-slab bearing on mass-concrete buttresses in extension of the pipe embedment in the adjacent drive. The base of the shaft, which is constructed monolithically with the corewall, was cut away so as to leave sufficient clearance above the chamber roof to allow of future corewall-deflexion. Details of the main access-shaft at the end of the 250-foot length of drain are shown in Figs. 17, Plate 1.

Shafts giving access to the system from the crest of the dam were constructed at chainages 1,050 and 1,950 (*Figs. 16*, p. 147).

The 2-foot-diameter outlet-pipe leads southward and westward from the main access-shaft, passing through three inspection-shafts and an ejector-pit constructed to enable the drainage to be discharged into the river in time of flood. The ejector-pit (Figs. 17, Plate 1) is divided into two compartments by a doubly-reinforced partition 7 inches thick, in the bottom of which is a flap valve so placed as to prevent flood-water from the river entering the ejector-well and the 2-foot-diameter outfall-pipe. The ejector-well contains four 3-inch ejectors fed from a 6-inch-diameter water-main from the reservoir, water from which is thus available to clear the well of drainage. With this arrangement the maximum head is available at the ejectors at periods of highest river-level.

The drainage normally passes into the outfall through the non-return valve, but in times of flood the valve may be closed and the drainage dealt with by the ejectors.

The Agricultural Drain.

This was laid down along the toe of the dam; it is, for the greater part of its length, 9 inches in diameter and follows the grade of the natural surface at a depth of about 4 feet. At the changes of grade and intermediate points, inspection-shafts 4 feet by 3 feet are installed with sumps to act as sand-traps. Prior to 1933 the pipe discharged

into the outlet-culvert at R.L. 717. With the construction of the needle-valves and the consequent closing of the culvert-mouth, the drainage was discharged to an outlet at R.L. 715 on the river bank.

Fig. 18.

JUNCTION OF COREWALL-DRAINS AND OUTFALL-TUNNEL AT INSPECTION-SHAFT
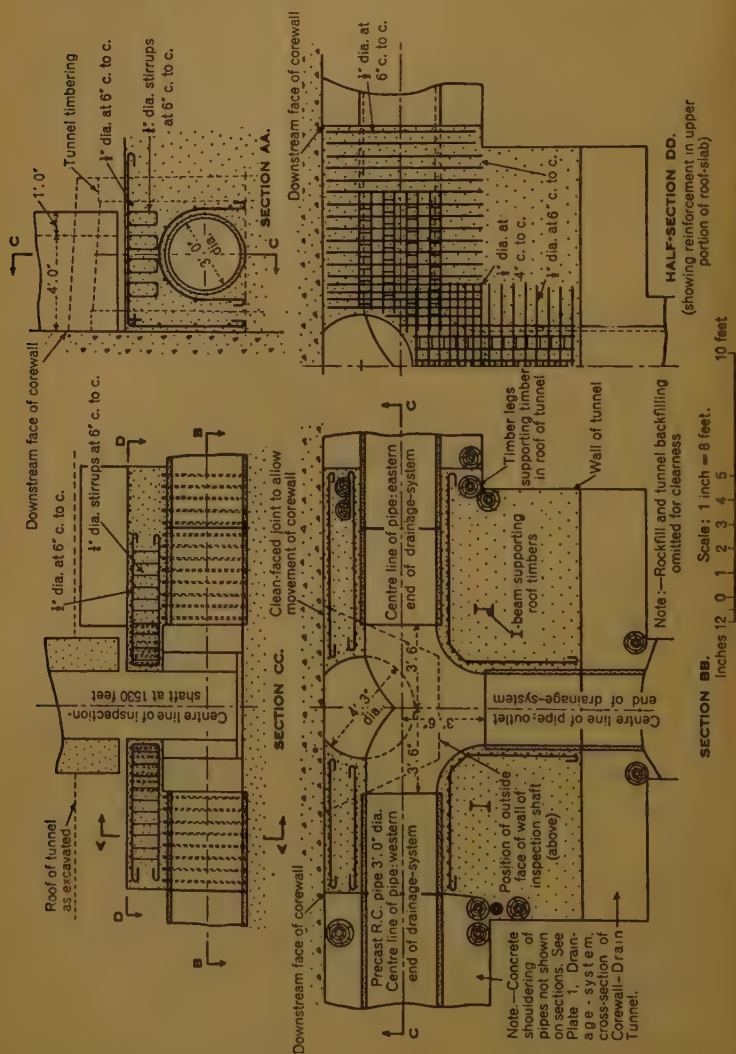
Construction.

The first step in the construction of the corewall-drainage system was to relieve the ground of as much water as possible to facilitate the driving of the tunnels. To this end a timbered shaft was sunk

at the lowest point of the rock surface, at 325 feet downstream of chainage 1,300. A well of 3-foot pre-cast pipes was constructed inside the shaft, and the space between the pipes and the timbering packed with spalls, the ground-water being pumped from the well. Further relief was obtained by continuous pumping at the inspection-shaft at chainage 1,530, where the water was eventually lowered to the level of the bottom of the shaft, the French drain at both the eastern and western section of the dam being thus gradually drained.

Simultaneously with this preliminary dewatering of the dam-foundations, the agricultural drain described above was laid; by February, 1930, the drain was completed for 1,990 feet to the junction-shaft at chainage 2,183 and the drainage discharged thence through the opening in the main outlet-culvert wall. During September–October, 1933, after the closing of the outlet-culvert by the needle-valve control-structure, the drainage was diverted to its final outfall (Figs. 17, Plate 1). Whilst the river can gain access to the junction-shaft, the agricultural drain is above flood-level throughout its length.

To prevent surface-drainage from isolated depressions on the hill-side below the contour-drain passing along the surface slope and thence under the dam, a 12-inch drain was laid in a small tunnel immediately beneath the natural surface under the rockfill at chainage 550 (Figs. 17, Plate 1). Rubble masonry walls were built up on the concrete floor on which the pipe was laid with open joints, and the drive backfilled with spalls. The drain discharged into the agricultural system at the adjacent inspection-shaft.

The construction of the main corewall-drainage system was begun in September, 1929. The main access-shaft, 9 feet by 5 feet clear inside the timbers, was first sunk, bedrock being entered for a depth of 9 feet to allow for a sump in the completed shaft. The drive to the corewall was commenced immediately on bottoming, and was 7 feet by 5 feet clear inside timbers. Ample ventilation was provided by the circulation of air through the rockfill when the corewall was reached.

As mentioned above, the drainage-system was designed so as to have the concrete work bedded on rock, variations in the surface-levels of which necessitated driving through depths of up to 3 feet above the tunnel floor. Where the rock dipped below grade level, a trench 2 feet wide was cut in the overlying gravel to permit of the base concrete being carried to bedrock.

Precautions were taken during the approach to the corewall to avoid the danger that the tunnel might be flooded by tapping possible " reservoirs " of entrapped water. Pilot-tubes 3 inches in diameter, one on either side of the tunnel, were driven forward for about 15 feet

ahead of the drive; although some drainage did enter the drive through the pipes, the pumping proved to have relieved the ground of much of the surplus water by the time the anticipated danger-zone was entered, and no large quantities of water were encountered. The gravel wash, however, was in a saturated state and water constantly seeped into the drive; all drainage was led to the main access-shaft, where it was pumped to the surface.

At the base of the shaft at chainage 1,530, from which point the tunnel branched right and left along the corewall, special timbering was used to provide space for the concreting of the junction-chamber. The approach-tunnel was here opened out to three times its width for a distance of 11 feet from the corewall and provision made to allow of the transfer of the load from the posts to steel struts, which were later embedded in the concrete of the chamber supporting the reinforced-concrete roof-slab (*Figs. 18*, p. 152). The clearance necessary for constructing the slab was provided by raising the tunnel-roof. The corewall-drives were similarly opened out for a distance of 11 feet on either side of the centre line, beyond which the original dimensions of 7 feet by 5 feet were adhered to in continuing the driving operations in an easterly and westerly direction, the upstream post resting against the corewall. The tunnel had advanced to the inspection-shaft by the 4th December, 1929, and driving along the corewall was commenced a fortnight later.

The clay and gravel wash of the approach-tunnel gave place to rockfill adjacent to the corewall.

On the east side, within 7 feet of the shaft at chainage 1,530, the leakage into the shaft was found to be due to cracks in the corewall; the 3-inch drain-pipe lying below the rockfill in the old French drain was uncovered and a small amount of drainage was found to be still passing through it. A similar condition obtained on the west side, the fill containing in each instance a good deal of slime and mud washed down from the overlying fill, which partly blocked the pipe.

Shafts giving access to the drainage-system from the crest of the dam at chainages 1,050 and 1,950 (*Figs. 16*, p. 147) were commenced in October, 1929, the timbering being semi-hexagonal in shape. Between December, 1929, and February, 1930, drives at the bottom of these shafts were opened out to join up with those extending from the junction at chainage 1,530. West of chainage 1,950 the drive was continued to chainage 2,196 where a mass of concrete was struck, and after entering this for a distance of 4 feet, it was decided to end the tunnel there. This was 50 feet short of the designed length, but included practically the whole of the affected portion of the corewall.

To prepare for the placing of the 12-inch reinforced-concrete pipes

tapping the rockfill over the eastern section of the corewall-drain, timbered galleries 3 feet by 2 feet were driven upwards at 45 degrees.

Concrete for the floor of the tunnel was composed of Delatite river sand and crushed stone from Sugarloaf, in the proportions of 1 cement, $2\frac{1}{2}$ sand, and 5 aggregate made up of 1 part of $\frac{3}{4}$-inch, 2 of $1\frac{3}{4}$-inch and 1 of $2\frac{1}{4}$-inch stone.

The seal at the western extremity of the tunnel between chainages 2,196 and 2,200 was first concreted, and the pouring of the floor commenced on the 11th April, 1930.

Pipe-laying was commenced at the western end (chainage 2,196) on the 7th June and at chainage 1,057 ten days later. Four 4-foot lengths of 3-foot pipe were placed at a time and the sides concreted; the night shift would then backfill with the broken rock and spalls, packing the latter tightly against the timbers. The 12-inch pipes in the " risers " driven to the French drain were connected to the main drain by means of cast-in-place reinforced-concrete junction-pieces.

After reaching the inspection-shaft at chainage 1,530, a short length of 3-foot pipe was laid in the approach-drive (from 250 feet left) for the purpose of forming a bond between that section and the concrete of the junction chamber. The base of the inspection-shaft (*Figs. 18*, p. 152) was demolished to give access from the crest of the dam to the corewall-drain and the approach-drive, and the pit-props supporting the roof-timbers were replaced with steel joists. The shaft was not underpinned, it being cast monolithically with the corewall and therefore of sufficient strength to support its own weight and the vertical load due to the pressure of the fill against it.

Before concreting the main access-shaft, the timber sets were covered with lagging, the space between this and the shaft laths being packed with rock. This method was employed in the case of all timbered shafts which were to be lined with concrete, with the dual object of reducing the quantity of concrete and, in the case of shafts in rockfill, of allowing settlement past the lining.

In concreting the shafts at chainages 1,050 and 1,950 cylindrical drop-bottom buckets of about 8 cubic feet capacity were employed. Placing the double-grid reinforcement in the confined space within the shaft-timbering proved a tedious operation. The bars were bent in the workshops to a schedule, but adjustments due to irregularities in the alignment of shaft and corewall often caused considerable delay.

Corewall-movement continued after a junction had been effected between the shafts and the main drain, but the provision of a slip joint between the shaft-lining and the corewall safeguarded the structure against fracture either throughout its depth or at its base.

In the work on the 2-foot-diameter outlet, the drives were 5 feet high with a 4-foot base and 2-foot roof and lightly timbered. Bedrock was followed to within 100 feet of the outfall, where sand was encountered and close timbering resorted to. One of the construction-shafts, 180 feet from the outlet, was widened during July, 1930, to form the ejector-pit previously described. The last 65 feet of the 2-foot drain was placed in a trench, and the work completed in September, 1930.

Conditions after Drainage.

The main sources of leakage through the corewall, as disclosed during the installation of the system, were fractures in the concrete immediately east of the inspection-shaft at chainage 1,530, and along the base of the corewall along the western section. A not inconsiderable contribution came up through the bedrock along the corewall, and, in the approach-tunnel, the seepage from the gravel wash formed an appreciable addition to the drainage.

The total flow, as will be seen from *Fig. 19*, ranged from 24 to 48 gallons per minute, and varied with the reservoir-level. Although this flow does not include the leakage through the cracks at the outlet-culvert, which never exceeded 20 gallons per minute, the whole quantity passing through the diaphragm of approximately 280,000 square feet in area is surprisingly small. Without adequate drainage, however, the cumulative effect of such leakage, as shown during the investigations immediately following the subsidence, would soon result in a condition of saturation endangering the stability of the bank.

The improvement in the condition of the natural surface since the drainage works is most marked, and observations from bores sunk since the completion of the work disclose a lowering of the ground-water level by about 5 feet.

Periodical gaugings are taken at a notched weir fitted into the 3-foot pipe at the access-shaft, and appear to indicate that the flow is subject to a seasonal variation from which no great departure has so far been noted. By means of chainages marked inside the pipe, any pronounced leak can be localized and kept under observation. It was for such inspection that the diameter of the pipe was fixed at 3 feet, which is the minimum size in which this can conveniently be carried out.

The drainage of foundations of this nature is a matter of some difficulty, but, with the combination of surface and bedrock drains as executed, the problem has been successfully solved, the entire system having functioned most successfully.

Fig. 19.

RELATION BETWEEN DRAINAGE-GAUGINGS AND RESERVOIR-LEVELS.
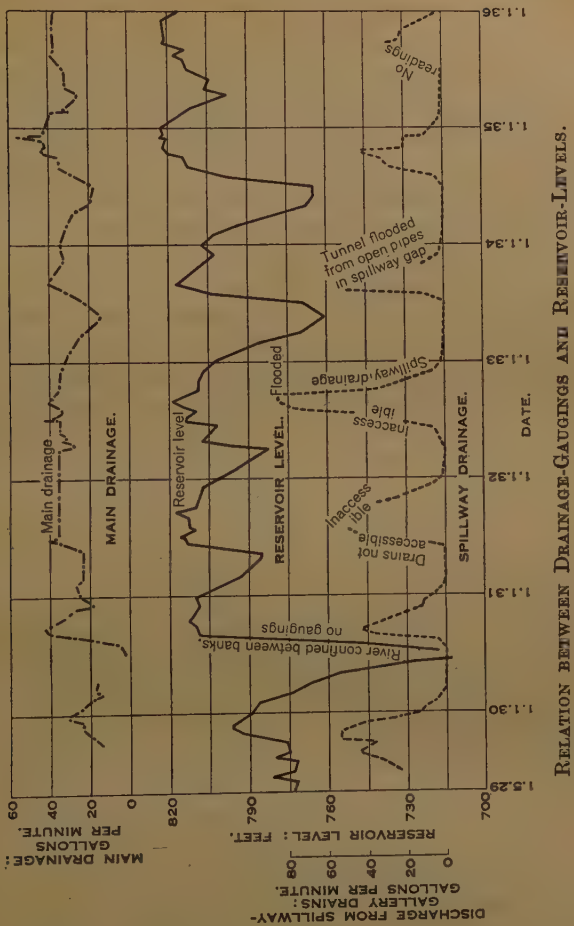
ORIGINAL MAIN OUTLET-WORKS.

Arrangement of Gates.

The release of water from the reservoir through the outlet-culvert was originally controlled by two tiers of four gate-valves, the upper at R.L. 754·75 and the lower at R.L. 706, as previously described (p. 121). Water admitted by the upper gates passed down to outlet-tunnel level through the four 4-foot 6-inch down-pipes, to the upper ends of which air was admitted through a vent-shaft 4 feet 3 inches in diameter extending up to R.L. 840. The original arrangement is shown in *Figs. 20* (p. 159).

11

Behaviour of Old Outlet-Control.

In the light of present-day knowledge of reservoir-outlets it is seen that no great efficiency could be expected from such an installation, even if the draw-off had been confined to the lower tier of valves. No provision had been made for admitting air immediately down-stream of the gates, and it is doubtful whether the vent-shaft (with the upper gates closed) was not actually detrimental to the hydraulic efficiency of the outlet, inasmuch as it acted as an inlet for air to be drawn into the conduit by a jet-pump action, resulting in surging of the water and consequent vibration.

With the upper gates in operation and the lower closed, it was found that air accumulated in the dead end between the junction of the down-pipes and the lower valves, and the pressure increased to such an extent that, at irregular intervals, water was ejected from the conduits with explosive violence, accompanied by heavy reverberations which shook the tower, tended to dislodge the lower gates, and endangered the system generally. This was due to water falling from R.L. 754 taking air with it from the vent-shaft, and thus acting as a form of hydraulic compressor. This occurred whenever the upper gates were opened more than 2 feet.

Some relief was obtained by leaving the lower valves open a few inches and thus providing a possible means of escape for the entrained air, which would be carried downstream by the jet under the gates ; this, however, did not completely overcome the trouble, and the lower valves suffered some damage. From 2 to 3 feet below the gates the sides of the valve-casings and circular pipes were badly affected by cavitation, the $1\frac{1}{4}$-inch metal having been eroded through to the concrete. The lifters were unable to operate the lower gates except under low head, on account of the damage to the sliding faces, and one of the gates could not be completely closed.

With the subsidence of the rockfill in April, 1929, and the consequent displacement of the outlet-tower, when its junction with the outlet-culvert was badly cracked (*Figs. 20*) the further use of this system, except in case of emergency, was considered undesirable. The power-outlet had been completed, and sufficient water for irrigation-requirements could be discharged through the turbines, the sill of the intake being at R.L. 754·5. The old outlet-system could thus be practically dispensed with, being only called into operation when the reservoir-level fell below that necessary to pass sufficient water through the penstock.

Recommendation of the Inquiry Board.

Complete control of the system of outlets was impossible, on account of the faults described above, and the following general

recommendations were made by the Inquiry Board for its improvement (see Figs. 21, Plate 2) :—

   (a) The cracks in the main outlet-tower should be repaired and the structure stiffened by means of a mass-concrete plug; the upper gates should be abandoned and air-vents provided behind the lower ones.

   (b) An extension should be made from the 4-foot 6-inch diameter cast-iron outlet-pipes from the tower through the outlet-tunnel.

   (c) Needle valves or other suitable valves should be placed on the ends of these extension-pipes at the downstream side of the bank.

   (d) Control-valves should be placed in the pipe-lines to supplement the needle valves.

Figs. 20.
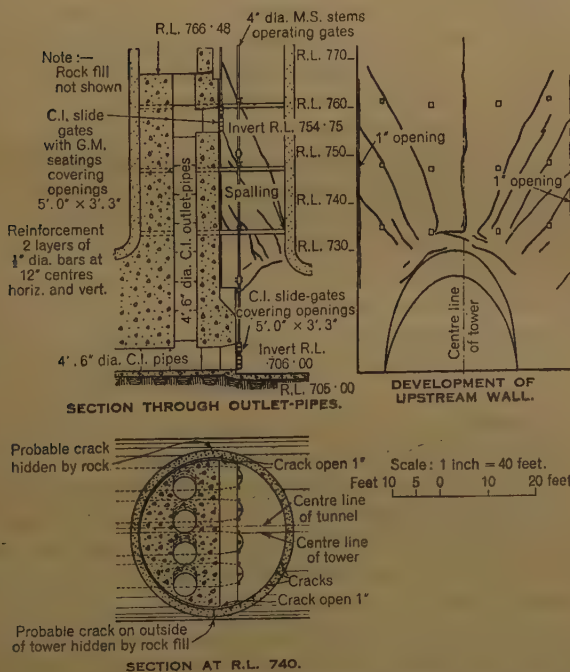
LOWER PART OF ORIGINAL MAIN OUTLET-TOWER, SHOWING CRACKS.

The upper valve conduits, down-pipes, chamber and vent-shaft should be sealed, and, on the upstream side, where the stabilizing concrete would be poured, 4-foot wells, one for each of the lower

valves, could be provided for the dual purpose of carrying the valve-stems and of giving access to the gate-valves for repair or replacement if necessary.

The means of supporting the additional mass was left to the Engineers of the Commission to determine, as was the design of the new outlets embodying the principles set forth in (b), (c) and (d) of the Board's recommendations.

RE-DESIGN OF MAIN OUTLET-WORKS.

Principles of Control of Discharge.

Considerable improvement has been effected in recent years in the design of high-pressure reservoir-outlets, arising out of investigations by the British firms, Messrs. Glenfield & Kennedy and Messrs. Blakeborough & Sons, Ltd., as well as by the Bureau of Reclamation of the United States of America.[1] Regulating devices for high dams have attained their highest development in recently-constructed examples in America. The original installation at Eildon was readily adapted to remodelling and reconstruction, and, except in minor details, particularly at the upstream end, a completely-modernized system has been installed.

In practically all dams projected at the same time as Eildon (about 1912), outlets were designed with upstream control; but with increase in heads the trend of development has been towards downstream regulation in an endeavour to eliminate the detrimental effects of the former method, due to the dissipation generally of energy within the conduits. While it is not proposed to go fully into the principles of high-pressure outlets, certain aspects of the question, as applicable to the reconstruction of the Eildon control-works, will be dealt with.

The gate valves, as originally constructed, were called upon to operate under heads of up to 70 feet; this has since been found to approach the limit for leaf gates under improved modern conditions, but the possibility of their successful operation under such heads appears to have been remote from the outset. The arrangement of the outlets in two tiers, with the object of keeping the pressures down to workable limits, resulted in a combination of circumstances which rendered them practically inoperative. The detrimental effect of the formation of a vacuum was also experienced in the outlets of the Pathfinder dam in America,[2] where, as at Eildon, no

[1] J. M. Gaylord and J. L. Savage, " High-Pressure Reservoir Outlets." Washington, 1923.   Elwood Mead, " Dams and Control Works "; pp. 92–103 (C. M. Day, " High-Pressure Reservoir Outlets ").   Washington, 1929.
[2] Elwood Mead, " Dams and Control Works," p. 93 (C. M. Day, " High-Pressure Reservoir Outlets ").   Washington, 1929.

provision was made for air-vents immediately below the gates. In this case an 8-inch air-vent branching into two 6-inch connexions is now used to supply air below a gate valve 5 feet square.

From a study of their performance generally, it became apparent that the Eildon gate valves should be used simply as sluices in the fully-open position or as emergency gates for the dewatering of the conduits or the protection of the valves downstream. As the size of the vent-pipe which could be introduced below the gate-valves was limited to approximately 6 inches diameter by the clearance between two steel joists embedded in the concrete supporting the lower valve-stem guide-block, it was doubtful whether an adequate supply of air could be admitted. However, during discharge under a 70-foot head since completion, a considerable improvement has been noted. The question of bell-mouthing the inlet to the conduits to effect a further improvement in the flow conditions was considered impracticable under the circumstances.

It was considered necessary to instal guard-valves in the new pipe-lines near the upstream end, immediately below the breeches-pieces, and butterfly valves were adopted, principally on account of their compactness.

Downstream control was to be effected at the outlets of the pipes by Larner-Johnston Type N. 78–68-inch needle valves, with hand-operated pilot-jet control.

Particulars of Remodelled Outlets.

The outlets are capable of discharging approximately 1,600 cusecs through each needle valve under a net head of 50 feet, this net head being the sum of the pressure and kinetic heads at the valve. The maximum head for which the valves were designed was 185 feet of water.

The 6-foot-6-inch outlet-pipes are of $\frac{1}{2}$-inch mild steel plate manufactured in 18-foot lengths made up of three plates with electrically-welded longitudinal lap joints. The circumferential joints are double-riveted butt-straps, and the pipes are supported on concrete saddles. Each pair of the 4-foot 6-inch conduits from the gate-well to the corewall (Figs. 21, Plate 2) is connected to the corresponding 6-foot 6-inch pipe by means of taper breeches-pieces 20 feet in length, special fitting-pieces fabricated by welding being required for adjustments of length and direction.

It was originally intended to embed only that portion of the breeches-pieces upstream of the fork, but later events proved the desirability of completely embedding the whole of the 14-foot tapered length in addition to the 6-foot fitting-pieces.

The butterfly valves in the outlet-pipes are spaced 18 feet apart

longitudinally to allow of placing the servo-motors in tandem between the pipes.

The needle valves are embedded in a breast-wall 5 feet 6½ inches thick closing the culvert and supporting the manual pilot-valve operating gear, and a rectangular well gives access to the culvert. The embedment of the valves was later extended 8 feet 6 inches upstream of the needle-valve casing to damp the excessive vibration experienced during the initial operations. From the breast wall a reinforced-concrete apron extends 15 feet 2¼ inches downstream, and supports three 18-inch piers slotted at their downstream ends to take a double row of 6-inch drop logs for use as cofferdams in dewatering the pressure-pipes. The whole structure stands on bedrock at about R.L. 703, and is designed to withstand the highest flood. No provision was made for the attachment of jet-dispersers, as it was considered that the river bedrock would resist scour (see p. 167). The new outlets are shown in Figs. 21, Plate 2.

RECONSTRUCTION OF MAIN OUTLET-WORKS.

Preliminary Work.

As a preliminary step in the dewatering of the main outlet-tunnel, the obstruction preventing the closing of one of the gates was removed by a diver in May, 1930.

In anticipation of the work in the tower, the sealing of the vent-shaft and down-pipes was commenced, as was the repairing of the cracks in the tower walls, which were all accessible at this time, as the reservoir-level had been lowered for inspection. To retain the concrete in the down-pipes during pouring a special form was fixed at the bottom of each pipe by the diver. By completely sealing these pipes the conduits would be without ventilation of any kind, and such advantage as was, at the time, thought to have been derived from the vent-shaft was preserved by the embedment of an 8-inch galvanized-iron pipe, carried up to R.L. 840, along the axis of each down-pipe. The upper gates were lifted clear of the openings and the intakes sealed; the chamber at that level was then filled and the concrete carried up the vent-shaft to R.L. 825. Close contact between the concrete seal and the interior of the chamber was ensured by grouting. It was thus considered that the horizontal cracks in the lower portion of each of the down-pipes (at R.L. 724) which had occurred as a result of the deflexion of the tower, were completely sealed. During subsequent work in the outlets, however, when the reservoir rose to chamber-level, water found its way into the vent-pipes in some manner and caused no little inconvenience

throughout the whole work. The concreting was completed in August, 1930. Cross bolts were left in the 8-inch pipes to hold the grouted plug.

Early in July, 1930, while the river was still low, the four pedestals at R.L. 722·5 were drilled and 6-inch galvanized wrought-iron vent-pipes grouted in for a depth of 1 foot 6 inches and carried up to R.L. 837. These were the largest pipes that could be introduced, and were of sufficient size to permit of $5\frac{3}{8}$-inch vents being subsequently core-drilled through the pedestals and the cast-iron valve-casings into the conduits 1 foot 9 inches below the gates.

Dewatering the Outlet-Culvert.

To effect a temporary closure of the mouth of the outlet-culvert, a cofferdam of some sort was necessary. In view of the irregularity of the bedrock and the necessity of complete reliability, a single-wall timber cofferdam connecting the wing-walls, and standing on a concrete base constructed on bedrock, was adopted. The concrete base could later be incorporated in the apron below the needle valves and further protection of the river-bed obtained at comparatively little extra cost. The dam was successfully completed and dewatered on the 27th May, 1931.

Demolition of Old Conduits.

With the dewatering of the main outlet-tunnel and the installation of electric lighting and air equipment, the demolition of the old reinforced-concrete ovoid conduits in the tunnel commenced. From three to five rock-breakers were used, and the work was completed in December, 1931, a total of 637 cubic yards having been removed, and the downstream ends of the cast-iron sections exposed. The average output per man per day was 1·00 cubic yard, at a cost of £2 16s. 0d. per cubic yard including all overhead.

The removal of the metal transition-castings, which commenced 6 weeks later, demanded the greatest care, as they were attached to the downstream flanges of the original 4-foot 6-inch outlets, which had to be preserved intact as they were the only means of connecting the new work to the old. Circumferential breaks were therefore made at the upstream ends of the transition-castings about 6 inches from the joints, to eliminate all danger of running cracks extending to them. To this end a ring of holes was drilled around each pipe, chases cut with pneumatic caulking tools between the holes, and the breaks effected by heating with an oxy-acetylene flame and quenching with water. The cast iron and surrounding concrete were then broken away. The remaining 6-inch length of the taper section was then unbolted, to effect which the tunnel-plug was

undercut.   Altogether about 43 cubic yards of concrete and 20 tons of cast iron were removed in demolishing the embedded conduits, the cost per unit of concrete and casting combined being £13 per cubic yard or about £12 per linear foot.   The removal was completed early in March, 1932.

Pipe-Foundation.

The excavations for needle valves and pipe-saddles were sunk through the culvert-floor to bedrock and about 100 feet length of the floor near the upstream end removed, where contact with rock was found to be faulty.   The water-pressure in the foundations was measured, the maximum uplift pressure recorded being equivalent to a head of 7·8 feet above bedrock.   Relief drains were left in this portion of the floor when concreting was completed.

Construction of Pipe-Line.

Lifting tackle had been erected at the mouth of the culvert for a live load of up to 10 tons, the heaviest parts to be handled being the breeches-pieces, which weighed $8\frac{1}{2}$ tons.   The steel pipe-lengths each weighed about $3\frac{1}{3}$ tons.

In April, 1932, the breeches-pieces were placed accurately in position, having been moved to the site on 6-foot-gauge steel bogies running on 60-lb. rails.   The dimensions of the fitting pieces were then determined so that the joint could be tightened without inducing strain in the old cast-iron pipe-flange.   Drainage-holes 6 inches in diameter had been cut in their undersides to pass the leakage from the gate valves.   (The discs cut from these holes were electrically welded back into position on completing the needle valves.)   As a safeguard against damage during discharge, a heavily-reinforced concrete slab, strongly dowelled to the culvert-plug, was cast round the junction of the cast-iron pipes and fitting pieces.

Each 18-foot length of the 6-foot 6-inch diameter pipes was lowered on to two 2-foot gauge steel bogies fitted with a bolster carrying a roller at each end, on which the pipe was rotated to the correct position before being moved into the tunnel.   Riveting of all the standard pipe lengths was completed on the 10th October, 1932, and measurements made for the closing lengths, which were received a month later.

The needle valves were assembled early in April, 1933, and the concrete work for the embedment was commenced immediately the final adjustments were made.   Both valves, after their initial discharge, were quite water-tight.

Emergency Discharge.

At this time the Goulburn valley irrigation-period had officially ended, and the country was experiencing a dry period. It became apparent that, failing rain within 3 weeks, it would be necessary to discharge water from the Eildon reservoir to supply an extra watering. The reservoir-level was at R.L. 762 (on the 26th April), and a discharge of about 600 cusecs was passing through the power-outlet. By the 10th May the level had fallen to R.L. 761, 6·5 feet above the sill of that outlet, and the discharge to 200 cusecs. It was considered desirable to arrange for discharge through the main outlet by about the 14th May.

The needle valves had not at this time been encased in concrete, and the construction necessary before further draw-off could be effected comprised the following operations :—rock excavation for apron-foundations, reinforcing and concreting piers and apron, placing a concrete apron between the cofferdam base and the downstream end of the pier apron, embedding the valves for the first lift, flooding the dam, and removing the diaphragm from in front of the valves. By the 29th April, the needle-valve embedment was completed to R.L. 715, and two shifts were worked to complete the work mentioned above. A diver was sent into the main outlet-tower to clear the gate valves of staunching material, the needle-valve plungers bolted back in the open position, and precautions taken to avoid the pressure building up inside the valves during discharge.

By the 11th May all was ready for the passage of water through the conduits. The pipes and valves had been securely anchored in their saddles, and the breeches pieces were resting on concrete blocks, which would be incorporated in the permanent anchorage and wedged in place against the culvert-wall. The impending discharge would be with upstream control, the water passing into a long conduit, and precautions were therefore taken to enable the valves to be closed without delay if necessary.

On opening the gates the water passed through the conduits with steady silent flow for about 30 seconds ; a series of violent detonations then commenced, and increased in intensity until the gates had been lifted to the half-open position. The indraught of air into the stream was then stopped by closing the 8-inch " vents " in the tower, which had been fitted with valves for the purpose of controlling the air-supply. This had the effect of quietening the detonations somewhat in the pipe, and of silencing them in the outlet-tower. Steady flow .was experienced at the outlet end, but detonations and severe vibration continued at the breeches-pieces ; air apparently continued to find its way into the pipe. This increased

to such intensity during further opening in an attempt to build up pressure in the pipe, that the gates were immediately closed.

It was decided to anchor the breeches-pieces to the arch of the culvert before continuing the tests, for which purpose a framework of 12-inch by 10-inch posts and beams was built up over them and wedged to the walls and roof of the culvert; the tests were then repeated; the detonation, surgings, and pronounced breathing of the pipe and needle-valve castings, however, continued. The tests, which were completed on 15th May, showed that steady discharge was impossible under the circumstances, and to continue would have resulted in severe damage to the installation. This bears out the experience at the Pathfinder dam previously referred to.[1]

No further discharge through the pipe was considered advisable until both the breeches-pieces and the needle valves were embedded in concrete suitably reinforced, and until downstream control of the discharge was possible with the completed needle valves. As it transpired, the situation in the Goulburn valley was opportunely relieved by the long-overdue rainfall, and the work on the outlets continued on the above lines until the 15th June, when the needle-valve assembly was complete and the control-structure ready for operation. The breeches-pieces had already been embedded in mass concrete reinforced by a 12-inch mesh of $\frac{1}{2}$-inch steel passing over each separately, but the needle-valve casings were still under observation, their embedment being subsequently completed in May, 1934.

Preliminary Discharge with Downstream Control.

With downstream control now available, the pressure was built up in the conduits and, at the end of June, 1933, a trial run was carried out under the new conditions; no provision, however, had been made at this stage for admitting air immediately downstream of the gate valves.

Considerable improvement within the pipes was noticed, but the severe detonations continued, and the breathing of the plates and needle-valve casing still occurred, for, although the entrances to the 8-inch " vent pipes " had been closed, air was being admitted through undiscovered fissures within the main outlet-tower.

Two out of every three saddles on the curved section were therefore carried completely around the pipes, the added upper portions being reinforced with four 1-inch diameter rods grouted into bedrock in holes drilled through the existing saddles and into the culvert walls.

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[1] See footnote 2, p. 160.

The discharge-regulators functioned well, minor adjustments only of the plunger control-valve being necessary to attain the balance.

Bed-Erosion.

From this time a series of discharges was carried out, and on each occasion masses of bedrock were dislodged, until a stilling pool was gouged from the river-bed. After the first 14 months, however, little further erosion took place.

STRENGTHENING MAIN OUTLET-TOWER.

Design Considerations.

The condition of this structure after the subsidence, and the initial step in its restoration as carried out during 1930–1931, have already been referred to (pp. 158 and 159). Before the new outlet-installation could be put into operation, the complete stabilization of the control-tower was essential. The principal steps in this work were to fill the tower with mass concrete up to R.L. 789 in order to strengthen it and increase its resistance to further overturning, supporting the concrete in such a manner as to leave unobstructed the sluiceways to the gate valves; to provide access through the mass to the valves by means of inspection-wells in which were supported the valve-stems; and to complete the drilling of the vent-pipes. The tower as completed is shown in Figs. 21, Plate 2.

As it was necessary to maintain the storage at a level above R.L. 800 for irrigation requirements, practically the whole of the work in the outlet-tower had to be done by divers.

The gate valves themselves were interchangeable, and as those in the lower tier had been damaged, they were replaced by the upper ones, after the gate-guides had been overhauled.

The concrete filling is supported by four piers placed eccentrically in the tower to conform to the disposition of the lower gate valves, three being constructed between the guide frames and the fourth (or "corner") pier against the eastern wall of the tower.

The design provided for welded mild-steel box forms in the shape of proposed piers with angle-iron frames and stiffeners, built up in three sections vertically between R.L. 705 and R.L. 719. (See Fig. 21, Plate 2.) The piers were 3 feet thick overall, and the forms arranged so as to permit of a diver working inside to assemble the parts and place the concrete.

The space between the piers and the base of the tower was closed by stiffened floor-plates, and a steel bulkhead.

The pipe wells were to be 48 inches in diameter. It was first proposed to use steel pipes in their construction, but the price quoted,

£50 per ton, was considered excessive and only the lowest 6-foot 6-inch sections were ordered in steel. Precast reinforced-concrete pipes were specified for the upper sections. Despite the saving in first cost by the use of the reinforced-concrete pipes, the extra cost of handling, fitting and placing under water more than offset the higher cost of the steel pipes, which could be lowered directly into the tower with little preliminary work. At a later stage in the strengthening operations a bid of £32 per ton was received for steel pipes. The advantage of using these had, by that time, been amply demonstrated, and for the last 24 feet of the concrete filling this class of pipe was adopted.

The steel pipe weighed 137 lb. per foot, compared with 467 lb. for the concrete pipes, so that 12-foot lengths of the former could be handled as conveniently as 6-foot-6-inch lengths of the latter, and the amount of diving work for a given length of pipe was greatly reduced.

The original plan of operations provided for the removal of the east–west channels (*Figs. 20*, p. 159), and the retention of the north–south joists until sufficient concrete had been poured to relieve them of any load due to the rockfill. The R.L. 734 series were to be removed first, after the concrete had been poured to R.L. 730, and then those at R.L. 747 and 760 to permit of concreting to R.L. 765.

After having reached this stage it was proposed to seal the wells with cast-iron covers, used some years previously on the original outlets, and then to dewater the tower above R.L. 765. In October, 1935, an investigation into the strength of the tower revealed that as a result of the cracking and deformation of the shell it would not successfully withstand the combined water and rockfill pressures, and the proposal was abandoned ; it was then decided to carry the concrete up (using steel pipe wells) to R.L. 789, and to lower the reservoir to R.L. 799 to permit of fixing the lower channel and guide-blocks (Figs. 21, Plate 2) in the dry, aiming to reach this level during February, 1936.

Plant Employed.

The concrete was mixed in two 7-cubic-foot mixers, and placed under water either from bags or bottom-dump 8-cubic-foot buckets. In each case it was stipulated that the container should be made to rest on the bottom before the concrete was released, and the raising of the open bucket (or bag) through the first few feet done slowly to prevent disturbance of the mass.

The same pneumatic drilling plant as used elsewhere on the works was employed under water ; both Holman and Ingersoll rotary drills, and rock-breakers or pneumatic picks manufactured by Holman and

by Hardy gave satisfactory service. These machines exhausted into water down to depths of 110 feet.

The diving equipment was mostly of Siebe, Gorman and Company's manufacture, with the exception of the actual dresses which were made by Heinke and Company. Manual pumps capable of supplying air to depths of 20 fathoms were used in the earlier stages of the work; the recognized decompression-periods were strictly adhered to throughout, and at times, while working at depths greater than 80 feet during continuous three-shift work (such as concreting) three divers would be submerged simultaneously, one working and two undergoing decompression.

The wages cost for the large number of men required at the pumps was excessive, and as a considerable saving could be effected by the installation of a reliable mechanical method of supplying air to the divers, the question of working from the compressed air service was considered. A trial air-control panel was assembled by Wilson Reid Pty. Ltd., a firm of industrial instrument makers of Melbourne. Air was led to the panel through a filter from a small receiver separated by a reducing valve from the larger receiver which also provided air for general use inside the tower. At the panel, two branch pipes led off through special control-valves to the divers' air-hoses. Each control-valve consisted of a $\frac{3}{4}$-inch globe valve modified by fitting it with a needle, or conical plunger, entering a rustless steel ring which rested in the original seating. The new full opening was $\frac{3}{8}$ inch in diameter, and, with the needle entered, gave a greatly reduced passageway. The pitch of the operating-spindle screw was reduced to 26 threads per inch and the rate of opening thus slowed down so as to necessitate a decided movement of the hand-wheel to pass sufficient air to the divers. A graduated disk and pointer were fitted to indicate the exact opening of the valve. Emergency manual pumps were connected through stop-valves to each diver's air-line. The attendants soon became expert in the use of the panel, and the divers evinced no desire to return to the manual pumps.

Oxy-Hydrogen Cutting Apparatus.

Underwater cutting was done, at all depths, by oxy-hydrogen apparatus.

Little was known by the industrial gas and torch manufacturers in Victoria of the art of cutting metals in deep water when this work was undertaken, although successful isolated cuts had been accomplished on harbour work in depths up to 30 feet or so. In removing the channel-irons from the main outlet-tower, an attempt was made to adapt the ordinary oxy-acetylene torch with a special tip to use in shallow water, but with indifferent success. Whilst

it was possible to use acetylene gas for preheating at first on the shallow cuts, it could not be employed at greater depths because of the danger of using the highly-compressed gas in the free or dissolved state.   Hydrogen was therefore substituted for acetylene and a special cutting-torch constructed which resembled the ordinary cutter, but which had a separate cutting-oxygen supply-line and a double cock for shutting off each gas without altering the pre-heating mixture as set by the torch needle-valves.

The question of lighting the cutting torch under water at the greater depths was important, for repeated delays would be very costly ; a small separate pilot torch was therefore devised to light the cutter. The blowpipe would not light from the pilot torch unless the correct mixture of oxygen and hydrogen were flowing to the cutting tip, and it was necessary to set this mixture by the blowpipe (or cutting-torch) needle valves at the surface before lowering the cutter to the diver.   To overcome the necessity of having to rely on a man at the surface gear to produce the proper mixture, a mechanical means of doing so was devised.   By experiment it was found that the ratio of hydrogen to oxygen in the pre-heating flame required for all depths from 13 feet to 90 feet remained constant at 3·4 of hydrogen to 1 of oxygen, and that the pressures varied equally in respect to each other, increasing in direct proportion to the depth.

The " Arnold " oxy-hydrogen cutting torch was then produced by the Australian Oxygen & Industrial Gases Pty. Ltd. of Melbourne, the needle-valves being replaced by set gas-jets supplying the correct proportion of each gas, and was tested by lighting-up and cutting at all depths down to the maximum available, 115 feet.   This torch (Figs. 22, Plate 2) has proved thoroughly efficient, and no trouble was experienced in lighting up and cutting at any depth within the range available.

The pressures required for working at various depths were as follows :—

Depth : feet.	Pilot oxygen and hydrogen.	Heating oxygen and hydrogen.	Cutting oxygen.
20	120	75	103
30	—	84	—
40	120	92	120
50	—	106	—
60	120	115	135
70	—	124	—
80	120	130	145
90	—	136	—
100	120	142	155
115	—	148	—

In actual operation, after fixing the cylinder-pressures and testing the flame, the blowpipe was turned off at the duplex cock and lowered to the diver, who would then relight it with his pilot torch.

The remainder of the work, done with this torch, was practically free from delay despite the awkwardness of some of the cuts.

The oxygen and hydrogen were compressed to 1,800 lb. per square inch, and it was possible to use 160 out of the 200 cubic feet content of each cylinder. The consumption of gas on some of the cuts is shown in Table XI.

TABLE XI.—CONSUMPTION OF OXYGEN AND HYDROGEN WITH IMPROVED BLOWPIPE.

| Details of cut. | Area of metal cut : square inches. | Consumption : cubic feet. | | | | | Remarks. |
| | | Pilot. | | Heating. | | Cutting. | |
		O	H	O	H	O	
12-inch by 6-inch by 54-lb. I-beam : average of eight cuts at 36 feet, four at 49 feet. Average time, 9 minutes.	16·04	6	7	19	92	58	
		0·4	0·5	1·2	5·7	3·6	Per square inch of metal cut.
4½-inch by 1-inch mild steel flat : average of two cuts at 60 feet, two at 80 feet. Average time, 1½ minute.	4·5	3	4	4	11	6	
		0·7	0·9	0·9	2·5	1·3	Per square inch of metal cut.

(NOTE : The steel had in each case been embedded in concrete, which was cleaned off as well as possible.)

Sixty cuts were involved in removing the beams and channels from the outlet-tower. The cost per cut using the original torch, and with manual air-pump for the diver, was £6 6s. 3d. ; using the improved torch, and with compressed-air supply for the diver, it was £3 11s. 9d. ; and in a test cut by hacksaw, with manual air-pump, it was £11 6s. 4d. These costs include all labour at the tower during the operation, viz. diver, attendants, pumpmen during the use of manual pumps, ganger, riggers and engine-drivers. The materials comprise gas and tools used in hand-cutting.

Practically the whole of the cutting was done in muddy water where the submarine lights were of no use, and where only a glow from the cutting flame served to illuminate the work.

Preliminary Work in Outlet-Tower.

In December, 1934, dismantling of the control apparatus in the tower was commenced. The hydraulic gate-lifters were disconnected

and removed, and parts of the timber deck and steel floor-beams were taken up to leave a hatchway sufficiently large to admit the largest pier-form.

The 4-inch diameter gate-stems and guide-blocks were next removed. Divers using pneumatic picks cut away the concrete encasing the channels and beams, and the steel-work was cut out by means of the oxy-hydrogen underwater cutting torch referred to above. The removal of the upper gates completed the preliminary work, the lower ones being left to close the outlet-pipes during painting ; they were eventually permanently replaced by the upper gates.

Pier-Forms and Wall Assembly.

The steel pier-forms and bulkheads were next placed in position under water ; to prevent movement during concreting the whole assembly was firmly bolted together and anchored to the tower floor and walls.

Concreting of the pier-forms was commenced on the 4th July, 1935, and was temporarily suspended at R.L. 721 to place 60-lb. steel rails in the mass for stiffening the overhang over the western (No. 4) gate valve. With the pressure of the rockfill on the tower walls now taken by the mass concrete, the 12-inch by 6-inch I-beams lying in a north–south direction at R.Ls. 734, 747 and 760 were cut out to make room for the reinforced concrete pipes, and these were raised to R.L. 765 in 6-foot 6-inch lengths, being secured in position in such a manner as to preclude any possibility of moving during concreting. During this operation preparatory work for succeeding operations was being done by two additional divers.

It was found practicable to set the pipes in position to within the nearest ¼ inch, and with the final placing of the gate-stems this degree of accuracy was proved to have been attained.

Concreting between R.Ls. 730 and 765 was carried out between the 24th and 30th October, 1935, in a continuous run, the surface at R.L. 730 having been cleaned off preparatory to pouring, and the concrete placed from bags in the confined spaces between the pipes and the south face wall.

Work Above R.L. 765.

An investigation was now made into the proposal to dewater the tower for the purpose of continuing the work under dry conditions. As a result, however, of an extension above R.L. 765 of the central crack in the north wall as reported by the leading diver, and of the general distortion of the tower due to its displacement, it was

determined that the shell in its weakened condition would not withstand the external water-pressure; the proposal was therefore abandoned and, at the beginning of October, the raising of the concrete a further stage to R.L. 789 was decided upon. For extending the wells 48-inch steel pipes in 12-foot lengths were adopted and, pending the arrival of the materials on the works, the gate valves were overhauled, and plant and tackle re-arranged for concreting.

Drilling Vent-Pipes.

A steam-driven core-drilling plant using chilled shot was installed in the tower at R.L. 840, and the $5\frac{3}{8}$-inch vent-pipes 1 foot 9 inches behind the gate valves, below R.L. 721, were drilled through 9 feet 4 inches of concrete and $1\frac{1}{2}$ inch of cast iron. To prevent the shot being lost while cutting through the sloping cast iron of the tapered entrance-piece, a hardwood block, shaped to the pipe, was jacked up against the casting; as the bit partially cleared the cast iron the shot was held in the cut in the hardwood until the metal was completely cored through.

Extending the 48-inch Wells.

The four remaining north–south I-beams at R.Ls. 773 and 786 were cut out with the oxy-hydrogen torch and removed to allow of placing the pipes. Steel pipes 12 feet in length with welded seams and flanges were received on the works early in November, and assembly was proceeded with concurrently with the core-drilling of the remaining vents.

Final Stage of Concreting, from R.L. 765 to R.L. 789.

The core-drilling plant was dismantled on the 5th December; the concreting was commenced on the 8th December, and carried to final level a week later. A total of 900 cubic yards of concrete was placed under water between the tower-floor level and R.L. 789. The mix used for this concrete was 1 : 1·7 : 3·4 (water/cement ratio 0·7 to 0·9), the coarse aggregate being screened and remixed for maximum density. Special attention was devoted to the careful placing of the batches by the divers and to the avoidance of churning of the surface. Segregation of stone was avoided by bringing up the level of the concrete evenly, to effect which divers before each shift were instructed in the approximate volumes of concrete to place within certain limits. Concreting proceeded continuously between the unavoidable breaks at R.Ls. 725, 730 and 765, at which levels the surface was carefully cleaned of silt before commencing the next pour. The amount of laitance formed was practically negligible.

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Placing Stems and Guide-Blocks.

The strengthening operation was thus complete, and the next step was the placing of valve-stem guide-blocks inside the gate-wells. These were secured by stainless steel bolts previously set in the tower concrete.

The water-level of the reservoir was lowered to R.L. 793 by the end of February, 1936, to enable the channels and guide-blocks at R.Ls. 812 and 799 to be placed in the dry. As the water subsided the gate-valve stems were lowered into position. By the end of February, 1936, the lifters and pump-connexions were replaced, and the channels and beams encased in concrete. With the embedding of the vent-pipes to R.L. 831 during March, the tower controls were ready for operation.

Grouting.

Mention has already been made of the leakage of air into the 6-foot 6-inch diameter pipes through the old mass concrete downstream of the dividing wall in the tower during discharge, and through the sealing concrete in the down-pipes and old vent-shaft into the 8-inch-diameter vents (p. 166). On completion of the strengthening of the tower, the outlet-pipes were dewatered and the vent-pipes grouted from inside the conduits. The grout, forced up from below, reached levels between R.Ls. 771 and 790, and the vents were then filled from the top to R.Ls. 825—827 at a pressure of 60 lb. per square inch. There is now no possibility of air being admitted to the discharge-conduits through any but the 6-inch vents placed immediately downstream of the gate-valves.

Discharge through the Remodelled Outlets.

No opportunity has yet occurred to test the new outlets under all heads. During discharge through the eastern pipe in June, 1936, with the reservoir-level at R.L. 778·25, 70 feet above the gate-valves, steady conditions of flow existed in the pipe from the breeches-piece to the curved length, where some vibration of a minor character occurred, being more noticeable between the hooped anchorages immediately over the saddles, at which points the top plates showed signs of slight movement. There was no indraught of air down the new vent-pipes in the outlet-tower. With the needle valve at 0·35 opening the jet was clear, showing no indication of explosions of entrapped air, and the new installation, as far as can be gauged from the tests to date, functions in a most satisfactory manner.

POWER-OUTLET.

During the original construction of the Eildon spillway an opening 19 feet by 16 feet was provided, its sill being at R.L. 756, and in 1925 this was adopted in the design of the hydro-electric plant as the inlet to the pressure-pipe (Figs. 23, Plate 2).

For the first 12 months after the subsidence the tipping of the rockfill on the upstream side of the dam had been proceeding mostly eastward of the line of the main outlet-culvert. During the low-reservoir periods in May–July 1930, concurrently with the work about to be described, the entrance to this outlet had been protected from being blocked by the rock tipped from the R.L. 824 berm, by building a rubble masonry wall over the coping. Tipping had thereupon been carried westward as far as possible, but, adjacent to the spillway, the old rockfill had already extended partly across the sill of the power-outlet so that further tipping became impracticable. It was, however, desired that an extra mass of bank material, to support the undisturbed clay at this end of the dam, should be placed with as little delay as possible. A protective structure or " blister tunnel " over the opening had therefore to be built. Another reason for its provision lay in the necessity for increasing the area of the trash-rack protecting the power-house penstocks. With the entrance of the intake moved westward to a point well clear of the fill, ample space would then be available to erect a structural-steel grid of substantial size.

With the view to inspecting the dam foundations and executing urgent work at lower levels, the reservoir was emptied after the irrigation-season in April, 1930. Storage would have to be recommenced, at the latest, in August, so that 3 months was the maximum period that could be anticipated in which to complete the protective structure. The highest level to which the storage could reach without interfering with the work was R.L. 750. The preparatory work commenced on the 4th May, when the water-level was R.L. 753.

With the limited working period, it was imperative that at least a 40-foot length of the blister tunnel should be completed before the rise in the reservoir. This would enable the rockfill to be extended and the adjacent portion of the bank strengthened, to some extent, in the event of the whole structure not being completed.

Blister Tunnel.

The blister tunnel, 74 feet in length, is designed, in section, as a rigid rectangular concrete arch, heavily reinforced, and dovetailed into chases in the spillway-face, the reinforcement being grouted

into the spillway-concrete (Figs. 23, Plate 2). The screens extend along the spillway for a further 40 feet, preserving the same cross-sectional area.

The concrete structure is designed to carry the rockfill, which slopes from the crest-level, R.L. 840, to the underside of the tunnel-entrance, the maximum depth of rock over the chamber being 25 feet.

For convenience in design the tunnel structure was divided into three sections, each length being reinforced for the maximum condition of loading obtaining in it. The main reinforcing-rods are set radially on 6-inch centres at the spillway-face and are $1\frac{1}{8}$ inch, 1 inch, and $\frac{7}{8}$ inch in diameter in the respective third sections.

The roof-slab over the original opening is supported by means of fourteen $1\frac{1}{4}$-inch rods, anchored to the upstream wall of the valve-well, 6 feet above the opening, by the embedment of the upper 5 feet of their length, bent down at a slope of 1 on 3. The lower ends are cogged round the lower $\frac{3}{4}$-inch reinforcement of the roof-slab, and, to ensure that all rods were equally stressed before the structure was called upon to take the load of rockfill, an initial tension was induced in each by means of turnbuckles into which are screwed the upper and lower portions of the anchor-rods. The curve of the chamber-wall at the entrance is of 9 feet radius, and at the western side of the opening the old concrete is cut to a radius of 5 feet.

Construction.

During the first week in May, 1930, the rockfill was cleared away from the emergency outlet, and a heavy timber retaining wall built to hold back the fill from the opening. The old protective grid of concrete-covered I-beams was removed, and the rockfill which had been tipped against the spillway face to R.L. 770 was removed to form a berm at R.L. 751 on which the work was carried out. As this had been settling for some years, it was considered firm enough to support the formwork.

By the third week in May the chases had been cut in the spillway-concrete and holes drilled on a slope of 1 in 3 for the embedment of the reinforcing steel.

Hardwood sills were bolted into the spillway-face to hold the formwork bracing timbers. The formwork for the floor was supported at R.L. 751 on 16-inch by 6-inch bearers supporting stringers, bearers and decking. Neat-cement grout was used to fix the rods, which were supported on timber bearers under their cogged ends until the remainder of the reinforcement had been placed and the

floor concreted. The inside-wall form studs were braced to the sills bolted to the spillway-face, and the roof-slab forms supported on knee-braced cross pieces. The remainder of the heavy reinforcement was now placed and grouted into the upper bonding holes, the turnbuckle tension-rods over the opening secured to the roof reinforcement, and the wall-slab reinforcing-steel completed. The wall and roof were then concreted.

By the 7th June, the eastern 40-foot section of the tunnel was complete. The tension-bars, however, were not tightened until the concrete was well hardened. On the 13th June the whole of the concrete work had been completed. The western side of the entrance to the penstocks was smoothed to a 5-foot radius and the tension-bars pulled up evenly. By the 10th July, 9 weeks after the commencement, all work on the intake-structure was completed.

Screens for the Power-Outlet.

The limitations of discharge due to the heading-up on the old screen protecting the penstock have been referred to on p. 122. The maximum discharge at no time exceeded 1,900–2,000 cusecs, at which discharge the pressure on the screen was considered dangerous. The new screen (Figs. 23, Plate 2) is supported on a structural-steel framework secured to the spillway-face by I-beam cantilevers embedded in the concrete, and provides ample waterway to ensure a low velocity of flow. The structure is 40 feet long and is divided into four 10-foot bays. The cantilevers and vertical beams carry the actual screens, which are made up of sixteen vertical grids and two sets of twenty horizontal grids placed on the side, top and bottom of the structure. They are of 3-inch by $\frac{1}{4}$-inch mild-steel flats spaced at 2-inch centres. The end panels support similar grids; all are clipped in place with U-bolts. The clear waterway is approximately 1,500 square feet.

The structural members were zinc-sprayed, but this proved to be useless as a protective agent under local conditions; rust appeared 12 months after placing. The cost of spraying was £7 per ton of structural steel.

On the 24th July, 1930, the erection of the trash-racks was put in hand. The cantilever-embedment holes in the spillway were cut out, and the cantilevers packed to the correct level, wedged into place, and the steel framework completely assembled by the 5th August. On this date the assembly work ceased, as the storage-period commenced. The top beams were concreted into their anchorages in the spillway and the work left until such time as an opportunity presented itself for casting in the lower beams and placing

the grids by diver. This was done in August, 1934. On the 1st of that month the holes were cleaned out in readiness for concreting the beams into place, which was done by placing hessian bags filled with cement (about three to the cubic foot) into the holes around the beams and ramming them tightly to force the cement through the hessian, thus enabling the bags to set into a fairly solid mass. A week later all the girders were embedded and the bottom grids placed, and by the 22nd August the whole work was complete. The screens had been coated with a bituminous paint over the sprayed zinc before placing.

Spillway.

The concrete gravity section constituting the spillway (Figs. 24, Plate 2), 682 feet in length, is constructed in the bed of the Goulburn river where the surface of the bedrock is R.L. 700, and is curved in plan, concave upstream, to the old western river bank where the structure is founded on rock at R.L. 780. From the end of this curve (radius 187 feet 9 inches at R.L. 808) a straight length skirts the edge of the old Sugarloaf quarry floor, which rises to its highest level at the upstream end of the by-wash where the rock shelf is at R.L. 808 ; here the line of the wall turns west to abut against the quarry-face.

A typical cross-section is shown in Figs. 22, Plate 2 (section BB) where the curtain-wall on the reservoir side of the structure is indicated ; on the downstream side of that wall holes were drilled with the intention of grouting the bedrock, some being left as pressure-relief drains to discharge water passing under the cut-off. The upstream gallery collects water entering directly through the construction-joints as well as through 6-inch diameter vents extending up to within a few feet of the crest.

On the two occasions on which the spillway was over-topped, the greater part of the flow passed along the unlined portion of the by-wash and carried away a quantity of rock in the form of boulders broken from the steeply-dipping strata. Prior to 1929 a portion of the channel was lined with concrete to prevent encroachment on the structure, and during 1936 further concreting was done across the whole width of the by-wash at R.L. 712, and along the edge of the spillway-steps up to R.L. 760.

The form of the original crest is shown in *Figs. 25.*

Since this work was designed, floods of unprecedented intensity have been experienced in the south-eastern portion of Australia, and the spillway-capacity has now been greatly increased. Mr. H. A. Hunt, the Commonwealth Meteorologist, has stated that a maximum

fall of 6 inches per day over the whole catchment of 1,500 square miles would be a " fair average " for which to provide at Eildon. Such a rainfall falling over the Eildon catchment would cause a flood which would have taxed the capacity of the original spillway to the utmost, raising the level of the impounded water to within 9 inches of the crest of the corewall, or 16·25 feet over the spillway.

The Board of Inquiry investigated the capacity of the spillway and determined that, in the light of the experience at Burrinjuck, this should be increased. Furthermore, the residents of the valley

Figs. 25.

ORIGINAL SECTION, 1929.

SECTION CUT DOWN IN 1930.

FINISHED SECTION WITH GATE, 1934.

LOCALITY PLAN.

SPILLWAY-CREST ALTERATIONS.
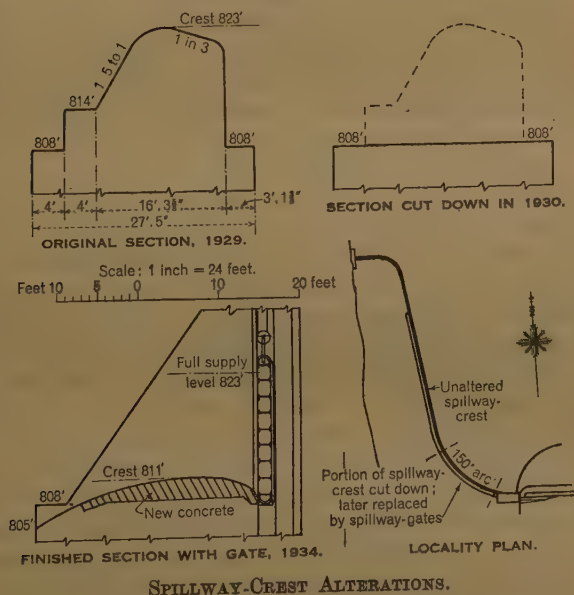
below the dam requested the Commission to lower the full-supply level, pending the restoration of the dam to a condition of safety. The request was acceded to and it was decided to cut a gap in the eastern end of the spillway-wall, extending to a depth of 15 feet below the crest-level (*Figs. 25*) for a length of 150 feet; this was capable of discharging approximately 20,000 cubic feet per second before the reservoir rose above the original full-supply level, and was later used as the basis of the design in providing extra capacity in the original spillway.

The scheme adopted was to construct in the gap six overshot sluice-gates (Figs. 24, Plate 2), each 20 feet long and 15 feet high, supported between and running in slots in the piers of a reinforced-

concrete control-structure. The combined functions of this structure are to restore the original full-supply level to R.L. 823 and to provide an efficient means of rapidly increasing the spillway-discharge when desired.

Description of the New Installation.

Each gate is 21 feet 11½ inches long and 17 feet 7 inches in overall height, the skin-plates being $\frac{7}{16}$ inch thick. The staunching arrangement is in the form of horizontal and vertical bars of No. 10 gauge gunmetal tube, the former 2½ inches and the latter 2 inches in diameter, with ends sealed by gunmetal plugs. To facilitate replacement, the staunching bar at the sill is held loosely in position by means of ten cast-iron supports set-screwed through the skin-plate to the lower cross-girder flange. Above this is a staunching angle shaped from 3-inch by 3¼-inch mild steel angle, having a cupped underface to hold the staunching bar in position, and bolted to the upper flange of the cross-girder. The vertical bars are held in a groove machined from a 12-inch by 3-inch mild steel bar shaped to form the roller-track.

The roller-chains contain fifty rollers 5 inches in diameter and 4¾ inches on the face, fitted with 1¼-inch gunmetal bushes running on stainless-steel roller-spindles 1 inch in diameter. The chain-links are of 2¼-inch by ¼-inch mild steel 11⅛ inches long. The down-stream roller-tracks are riveted to the flanges of the vertical end members of the gate-frame ; on the upstream side guard-channels 6 inches by 3 inches are riveted to the horizontal I-beams.

The gate-sills are on the chord of a circular arc of 189 feet 7 inches radius and overhang the upstream face of the spillway. They are " ⌐ " shaped in section, 10½ inches in height, with a machined vertical face, 7½ inches in width between bevelled edges, serving as a seating for the staunching bar in the closed position of the gate. In plan the gate-guide castings are channel-shaped, with 13½-inch back, and sides 8⅞ inches and 5⅝ inches in depth ; they extend from R.L. 784 to R.L. 831. The lowest 9-foot lengths of the roller-tracks are cast with the guides, but the upper lengths are in the form of separate rocker track-castings with a face 5¼ inches wide, and are free to adjust themselves to any movement under the loaded roller-chain. From R.L. 831 the guides extend up to R.L. 839·92 as plain flat castings or extension-plates.

The hoisting equipment for each gate consists of two winches, one over each end of the gate, driven through reduction-gears and shafting by a 7½-h.p. 3-phase motor placed between them ; the gates can be operated either singly or all together. Provision is being made to lift the gates above deck-level for overhaul when necessary.

The gates run in slots in 6-foot-thick piers erected on the R.L. 808 level at 25-foot 8½-inch centres on the line of the sill-casting, rising to R.L. 837·21 (the underside of the deck-beams). Below R.L. 808 the piers overhang the spillway-face and extend downward to R.L. 778·47. It is thus possible either to lower the gates about 6 feet below sill-level or to lift them clear of the structure.

The overhanging portion of each pier below R.L. 808 is anchored to the old concrete by rods grouted into holes drilled in the upstream face of the wall, and the base of each pier, between R.Ls. 778·5 and 784, is keyed into the wall by means of a rectangular chase 6 inches deep, with grouted cogged steel dowels. At pier No. 1 a reinforced curtain-wall 2 feet thick standing out 11 feet 3 inches from the spillway-face extends downward to the roof of the blister tunnel to hold back the rockfill of the bank and prevent it from encroaching on the gate-guides of that and the adjacent pier. Bond with the blister-tunnel roof is prevented by means of a " malthoid " strip, allowing free deflexion of the curtain-wall.

It was originally intended to construct the piers of plain concrete, but after the construction of the first lift of pier No. 5, following a flood period (August–November, 1933) during which hair-cracks developed as a result of the unequal contraction of the concrete, the design was revised and reinforced-concrete piers substituted. The reinforcement consists of a grid surrounding the pier, composed of ⅝-inch bars spaced at 8-inch centres vertically and 14-inch centres horizontally, with 3 inches of cover, the vertical bars being grouted to a depth of 40 diameters into the old work. The deck is of reinforced-concrete slabs, supported on concrete-encased I-beams.

The level of the spillway-crest between the piers has been raised to R.L. 811 by means of a curved reinforced-concrete slab secured to the concrete at R.L. 808 (*Figs. 25*, p. 179). A section of this crest normal to the sill is elliptical in form for the upstream 6 feet 9 inches, the downstream part of the section being a vertical curve of 37 feet radius. The downstream portion of the crest, between R.Ls. 808 and 805, is cut out of the old concrete, a 6-foot width of the original concrete being left intact at the base of each pier. The raised crest is reinforced with a grid of ½-inch bars at 9-inch centres, with 3 inches cover, and is anchored by ⅝-inch bars at 3-foot centres grouted into the old concrete, their cogged ends embracing the uppermost rods of the reinforcing grid.

Constructional Operations.

In September, 1929, pneumatic picks or rock-breakers were employed to cut down the 150-foot arc of the existing spillway-crest to R.L. 808. By the end of October, the gap was completed, a total

quantity of 1,035 cubic yards having been cut out at an average rate of 0·80 cubic yard per machine-day of 7½ hours.

In December, 1932, the work of installing the sluice-gates and control bridge was commenced, the first step being to increase the length of the gap by 8 feet. The cutting of 70 cubic yards necessary for this was completed, during the ordinary day work, in January, 1933 ; the average output per machine-day of 8¾ hours was 1·26 cubic yard, an increase of 40 per cent. over that of the three-shift work carried out in 1929.

The construction of the overhanging portions of the piers below R.L. 808 was planned to enable their completion with one shift during the lowering of the reservoir, and before the flood period commencing in August or September, 1933, during which the preparation for the pier and deck work above R.L. 808 would be completed. Actually, the gap in the spillway wall was submerged in mid-August of 1933.

Concurrently with the extension of the spillway-gap in December, 1932, the drilling of the holes in the crest at R.L. 808 for the anchorages of the pier and curved crest was completed. The work was done from rafts at the outset, and, as the reservoir-level subsided, during the irrigation period, hanging platforms were employed, these and the pneumatic tools being suspended from the gap.

The different operations of the pier-construction proceeded successively from the western end at No. 7 pier. Chases were first cut to accommodate the gate-guides and sills with their connecting bolts ; pier anchor-rods, scaffolding and form-bolts were grouted into position, and scaffolding erected ; pier-form yokes and panels were then placed and the pier concreted.

The work at pier No. 1 was complicated by the depth of rockfill over the tunnel protecting the emergency outlet, the roof of which had to be exposed by sinking a timbered shaft 12 feet by 12 feet through the 1-in-1·4 slope of the fill, the uneven pressure of which necessitated heavy external bracing to the face of the spillway wall, and diagonal struts inside the shaft.

The downstream side of the crest was cut to shape with pneumatic tools, the slight irregularities being filled with a cement wash, which was subsequently found capable of resisting the action of the water in the nappe.

It was possible to reduce the weight of the formwork-timbers for the overhanging pier-sections by first pouring the lift between R.L. 778·47 and R.L. 785·5 and allowing it to set for some days ; this was used as a bracket to support the weight of the concrete immediately above it and thus relieve the scaffolding.

The first lift above R.L. 808 in piers Nos. 5 and 6 was poured to

R.L. 816 before the high-reservoir period which caused the suspension of work in August. No. 1 pier, constructed inside the 12-foot by 12-foot shaft, was treated separately and the concrete poured ahead of the other piers in order to allow the shaft-timbers to be drawn and the curtain-wall to take the weight of the rockfill.

As a result of the formation of contraction-cracks in the bases of piers Nos. 5 and 6, it was decided to reinforce all piers with a vertical grid with 3 inches of cover. Before proceeding with their construction, four vertical holes were drilled on either side of the cracks from R.L. 816 into the floor of the spillway-gap to R.L. 806, and 1-inch bars grouted in to bind the piers together. A line of holes at 14-inch centres was drilled around each pier at R.L. 816, 6 inches in from the face, to take the vertical rods of the reinforcing grids; these, on grouting, were bent to their correct position on a slope of 6 to 1, and carried vertically upwards 3 inches below the pier surface, ultimately to the deck level.

A complete set of formwork having been prepared for each pier, concreting proceeded in rotation to R.L. 837·21, the level of the underside of the steel deck-beams.

The deck-forms were hung from the steel deck-beams, and test loads were applied to ascertain the allowance to be made for deflexion during concreting.

The curved crest-slabs were next concreted, the old surface at R.L. 808 being cleaned and washed with neat cement grout to obtain as close a contact as possible. A comparison of the old and new crests is shown in *Figs. 25*, p. 179.

A working platform was rigged for the deck work, and the first steel joists were placed at the end of March, 1934. These were connected by rigidly welding on channels and secured to the piers, and the deck formwork commenced early in April. The deck-slab reinforcement was first assembled in the shops and each bar marked to ensure accurate placing.

The gates had each been received on the works in two sections, and were transported by gantries and skids to No. 1 bay, whence they were lowered by cableway on to bogies, and moved to their respective positions on to I-beam supports overhanging the deck curbs for assembly.

In hoisting the gates over the slots twin towers of braced Oregon timbers were employed. These were 27 feet in height, with an I-beam spanning the gap between them, and were erected on the deck, one on either side of a gate, alternate gates being assembled to allow room for towers in the intermediate deck-spaces. A self-supporting gantry of this nature was necessary on account of the narrow deck and the impracticability of guying either derricks or

shear-legs at such a height above ground-level. The gates were then lowered into their guides, where they were temporarily suspended below deck-level.

The assembly of the deck winches and motors was completed by the 5th September, 1934, and the control-gear was housed in the power-outlet tower. The cables were tested and the gate-position indicators and control-switches adjusted, and the reservoir-level was controlled for the first time on the 18th October, 1934.

Spillway-Capacity.

Discharge-curves for the original spillway, and for the altered condition of the crest both during construction and after completion to the new design, are shown in *Fig. 26.*

Fig. 26.
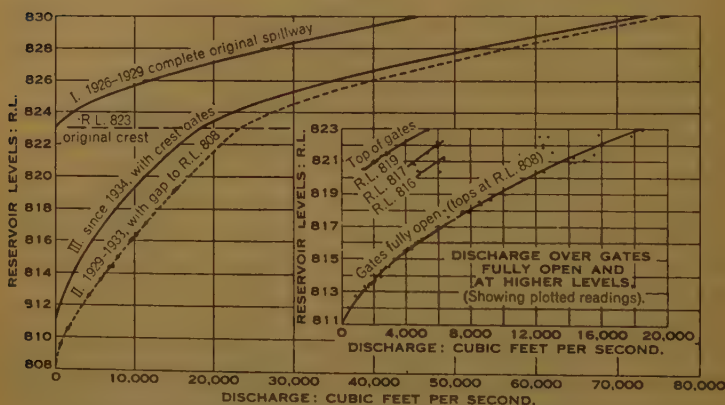
MAXIMUM DISCHARGE OF SPILLWAY AT VARIOUS STAGES OF
RECONSTRUCTION.

Curve I gives the discharge over the original ogee-section spillway-crest 672 feet long at R.L. 823, calculated from Horton's formula $Q = CLH^{\frac{3}{2}}$, where Q denotes the discharge in cusecs, L the length of the crest in feet, H the head in feet, and $C = [3.62 - 0.16(S-1)]H^{\frac{1}{2}}$ (S denoting the cotangent of the slope of the upstream side of the weir crest).

Curve II gives the discharge over the spillway with the gap 147·5 feet long cut to R.L. 808. From analyses of tail-gauge discharges below R.L. 823, the coefficient C in the formula $Q = CLH^{\frac{3}{2}}$ was found to tend with increasing values of H towards a value of 2·7. This coefficient was used to extend the discharge-curve above

R.L. 823, and the discharges thus found added to the discharge over the residual unaltered spillway calculated from the formula for curve I.

Curve III gives the discharge over the completed spillway with six 20-foot gate-openings having crests at R.L. 811 shaped to the nappe for R.L. 823. From analysis of tail-gauge discharges below R.L. 823, the coefficient C in the formula $Q = CLH^{\frac{3}{2}}$ was found to be $3\cdot3 + 0\cdot029H$. That formula was then used to determine gate-discharges for water-levels above R.L. 823, which were added to the discharge over the residual unaltered spillway calculated from the formula for curve I.

The inset diagram in *Fig. 26* gives the measured discharges over the tops of the gates at various openings.

Gates in Operation.

The behaviour of the gates when being operated under pressure is characterized by their smoothness of movement and complete control at all levels. The time occupied in raising a gate the full distance of 15 feet is 4 minutes. The coefficient of friction of a gate in operation, with reservoir-level at R.L. 822·75 (3 inches below full-supply level) was found to be : at start of movement 0·075, while running 0·052. The staunching bars are very effective.

With the new curved crest at R.L. 811 the nappe leaps the upper steps in the downstream face of the gravity wall; at the western end it strikes the apron at R.L. 782, while, at the three gates near the power-outlet tower, the water impinges on the lower steps which break up the nappe and serve as energy-dissipators, which, together with the water accumulating on the apron at R.L. 712 acting as a further aid to the stilling effect, reduce the velocity of discharge from the spillway to a comparatively low figure.

The gate-installation as a whole is proving most satisfactory in the regulation of reservoir-level.

Lining By-Wash.

This work comprised coffer-damming the river-bed below the by-wash by means of a low sandbag and clay-puddle dam, dewatering the river-bed, excavating keys for the concrete steps from the rock by pneumatic drills and rock-breakers, dowelling, placing formwork and concreting. Excavated rock was used as plums in the concrete, of which the total amount placed was 1,038 cubic yards.

MATERIALS.

Rockfill.

The rock was obtained from the Silurian formation at each end of the bank; that from the Sugarloaf quarries was mainly hard dense slate, whilst that from the Pinniger side of the valley was generally soft shale with some sandstone. The weight of the rockfill mass is about 90 lb. per cubic foot.

Clay.

The puddle core is composed of clay taken from borrow-pits, samples of which were subjected to tests for the purpose of comparing their power of resistance to immersion, and their " toughness." Reference has already been made (p. 126) to Professor Chapman's experiments made on samples of the actual puddle.

In the pit samples the moisture-content in the natural state was determined; the clay was then thoroughly dried and ground to a powder, that passing through a No. 10 sieve (10 meshes to the inch) was then mixed with water ($1\frac{1}{2}$ lb. clay to 8 oz. water in each case) to a workable consistency, and the following tests were made :—

 (1) " Ball " test. A ball 3 inches in diameter was immersed in water and the period noted during which it resisted disintegration.

 (2) " Roll " test. A roll about 2 inches in diameter was broken in tension and the nature of the break noted.

 (3) " Saucer " test. The sample (2 lb. of clay with 8–10 oz. of water) was moulded into a saucer 6 inches in diameter with bottom and sides $\frac{5}{8}$ inch thick. The mould was of machined mild steel, giving a smooth uniform finish. The flaking of the surface on filling the saucer with water was noted as a guide to resistance to disintegration, and the cracking of the rim as an indication of shrinkage.

The " clay " content was determined in each case, and was as shown below.

Sample No.	Loose dry weight; lb.	Clay-content; per cent.	Moisture-content: per cent.
" Magazine " pit M1 .	72	69	17
M2 .	74	74	16
M3 .	72	82	16
East end pit E1 . .	63	75	14
E2 . .	64	80	17
E3 . .	63	81	19

The results of the three tests were as follows :—

(1) Ball test.

M1 and M2. Commenced to flake as soon as placed in water ; top had flaked away after 4 hours and collapse was complete after 8 hours.

M3. As in M1 and M2 ; initial and final disintegration in $3\frac{1}{2}$ and $7\frac{1}{2}$ hours, the residue being of silty grains spreading out to a flatter angle than M1 and M2.

E1, E2, and E3. Initial flaking in 2, $2\frac{1}{2}$ and 3 hours, final collapse of sample in 5, $4\frac{1}{2}$ and 5 hours respectively.

(2) Roll test.

M1, M2, M3, E1 and E3 gave a short " strong " break, E2 only showing a " weak " break (tensile strength not measured).

(3) Saucer test.

M1 (2 lb. dry clay, 9 oz. water). Flaking of surface commenced on saucer being filled with water. After 2 hours large crack formed over full depth of rim.

M2 (2 lb. dry clay, 8 oz. water). Flaking on filling. Large rim crack after $2\frac{1}{4}$ hours.

M3 (2 lb. dry clay, 8 oz. water). Commenced to flake on being filled with water, which had all soaked through the bottom after 18 hours ; no cracking was noticeable.

E1 (2 lb. dry clay, 10 oz. water). Very slight flaking was noticeable on filling with water, which seeped slowly through the bottom after 15 hours.

E2 (2 lb. dry clay, 10 oz. water). No flaking ; water lost through cracking of rim in 14 hours.

E3 (2 lb. dry clay, 10 oz. water). No flaking ; water lost through rim cracks and bottom seepage.

Samples of clay from bores in the puddle-wall were tested for plasticity, an index to which was obtained by measuring the diameter of indentation produced on a flat surface of clay by dropping a steel ball of $1\frac{5}{8}$ inch diameter weighing 0·632 lb. from heights of 12 inches and 24 inches, the clay being moulded, for the purpose, into cylinders 3 inches in diameter and $1\frac{3}{4}$ inch high.[1] The moisture-content of a number of samples was determined by drying to constant weight at 100° C. The test report states that " many of the samples contained a number of small stone fragments and while the tests were directed towards the determination of certain properties of the clay,

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[1] This test is given by Mr. A. L. Bell, in "The Lateral Pressure and Resistance of Clay, and the Supporting Power of Clay Foundations." Minutes of Proceedings Inst. C.E., vol. cxcix (1914–15, Part I), p. 264.

and although the influence of these fragments was eliminated in the main, it is possible that they may have contributed some slight irregularities to the observations recorded. It is evident from the results that clays of varying characteristics are represented and the plasticity as exemplified by the drop test is dependent on other factors besides the water expelled at 100° C." Specific-gravity determination on four specimens gave figures ranging from 2·02 to 2·15. The moisture-content ranged from 16·4 per cent. to 25 per cent. of the wet weight. The results of observations on samples from three bores at chainage 1,550 and distances of 5 feet, 45 feet and 75 feet from the corewall are shown in Table XII.

TOTAL COST.

The work was practically completed in July, 1936, and the total expenditure was as follows :—

	£
Embankment	206,066
Spillway, including by-wash	35,530
Drainage	35,774
Outlet-works	58,320
Temporary works, workshops, camps, etc. . . .	20,293
Engineering surveys and investigations	24,228
	£380,211

All local overhead costs are included in the above, but head-office charges (insurance, interest, etc.) are not included.

CONCLUSION.

On the 13th August, 1936, the Inquiry Board reported that they were satisfied with the condition of the dam after restoration.[1]

The work described in the Paper was carried out on the recommendation of the Board of Inquiry by direction of the State Rivers and Water Supply Commission, under the Chairmanship of the late Mr. R. H. Horsfield, M. Inst. C.E., to the designs of the then Chief Designing Engineer, Mr. W. A. Robertson, M.C.E., M. Inst. C.E.

The remedial operations were instituted by Mr. C. P. Farie Wright, M. Inst. C.E., immediately after the subsidence. The Author was Executive Engineer resident on the works from August, 1929, until their completion in 1935.

The engineering construction staff comprised Mr. H. H. Williams,

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[1] A copy of the Report accompanies the MS. of the Paper, and may be seen in the Library of The Institution.—SEC. INST. C.E.

TABLE XII.—SAMPLES OF CLAY FROM COREWALL.

Bore No. 1, 5 feet from corewall at chainage 1,550 feet. Boring commenced R.L. 817; clay entered R.L. 784·25.

Sample No.	R.L. of sample.	Colour.	Consistency.	Diameter of impression in drop test: inches		Moisture-content: per cent. of wet weight.	Specific gravity.	Remarks.
				12-inch drop.	24-inch drop.			
1	784·25	Varies grey to brown.	Stiff—almost granular.	0·97	1·10	19·1	—	First clay struck.
2	779	Varies grey to brown.	Stiff—almost granular.	1·01	1·12	—	—	—
3	774·50	Varies grey to brown.	Stiff—almost granular.	0·94	1·05	17·5	—	—
4	766·50	Lighter than 3.	Stiff—almost granular.	0·94	1·10	—	—	Squeezed into hole over-night.
5	approx. 762	Uniform light red.	Soft plastic.	1·33	1·48	—	—	—
6	759·75	Uniform light red.	Fairly soft plastic.	1·20	1·36	—	—	Squeezed into hole over-night.
7	777	Uniform light red.	Very soft plastic.	1·34	1·50	21·0	—	Some vegetable matter.
8	755·75	Uniform light red.	Soft plastic.	—	—	—	—	—
9	750·75	Yellowish-brown.	Soft plastic.	—	—	—	—	Very uniform smooth pug.
10	746	Yellowish-brown.	Slightly stiffer than 8.	0·98	1·12	—	2·02	Very uniform smooth pug.
11	743	Yellowish-brown.	Soft plastic.	—	—	20·1	—	Very uniform smooth pug.
12	742	Yellowish-brown.	Soft plastic.	—	—	19·4	—	

13

TABLE XII (continued).

Sample No.	R.L. of sample.	Colour.	Consistency.	12-inch drop.	24-inch drop.	Moisture content: per cent. of wet weight.	Specific gravity.	Remarks.
				Diameter of impression in drop test: inches.				
Bore No. 3, 45 feet from corewall at chainage 1,550. Boring commenced at R.L. 810; clay entered at R.L. 777.								
20	775·5	Varies grey to brown.	Soft but granular.	—	—	—	—	—
21	766	Varies grey to brown.	Soft but granular.	1·05	1·18	18·1	2·10	Some grit.
22	—	Varies grey to red.	Stiff granular.	1·01	1·12	—	—	—
23	755	Grey and brown.	Stiff plastic.	0·99	1·17	—	—	Not uniform.
24	751	Light brown.	Soft plastic.	—	—	—	—	Smooth uniform pug.
25	745·25	Light brown.	Stiffer than 24.	1·07	1·24	18·4	—	Smooth uniform pug.
26	741	Light brown.	Fairly soft plastic.	—	—	—	—	Mixed with small water-worn pebbles. This is apparently natural surface.
27	740·25	Light brown.	Fairly soft plastic.	1·03	1·20	—	—	
28	738·50	—	—	—	—	—	—	No sample.
Bore No. 6, 75 feet from corewall at chainage 1,550. Boring commenced at R.L. 801; clay material entered at R.L. 766.								
37	766	Light red.	Soft plastic.	—	—	—	—	Clayey material with much grit and fine stone.
38	764	Light red.	Soft plastic.	1·19	1·36	—	—	Mixed with grit.
39	758	Red.	Firm plastic.	—	—	—	—	Mixed with grit.
40	754–5	Light red.	Very soft, almost slurry.	—	—	—	—	Clay mixed with stone.
41	753	Yellow.	Soft and wet.	1·17	1·30	—	—	—
42	750	Red to yellow.	Soft granular.	1·27	1·40	—	—	—
43	748	Greyish brown.	Soft and moist.	—	—	15·6	—	—
44	741·5	Yellowish clay and grey silt.	Soft plastic clay and wet friable silt.	—	—	—	—	Apparently at natural surface.
45	739·5	Light red.	Stiff plastic.	0·98	1·12	19·4	—	Below natural surface.

B.C.E., who was Assistant Engineer from the commencement until 1932, and Messrs. H. L. Byham, L. J. Scott, and C. L. Sanders, Assoc. M. Inst. C.E., Shift Engineers; Mr. Byham later acted as Assistant Engineer until the appointment in 1934 of Mr. C. T. Stafford, B.C.E., to that position.

The Paper is accompanied by forty-eight sheets of drawings and twenty-one photographs, from some of which Plates 1 and 2, the Figures in the text, and the half-tone page plate have been prepared.

Discussion.

Mr. W. Binnie.   Mr. W. J. E. BINNIE, Vice-President, remarked that the failure described in the Paper was one which might occur to the work of any engineer, and he wished to express his sympathy with those who had designed the original works. The Paper was of far more value, as describing a failure, than Papers which dealt merely with success-ful works, and it contained numerous points of interest. In 1881 Sir Benjamin Baker had read a very interesting Paper [1] in which he described the behaviour of retaining walls of many types. No less than 9 miles of such walls had been constructed for the under-ground railways of London, some of them of very great height, and Sir Benjamin described what had happened to them. Some of them, though apparently far too thick, had failed ; some of them apparently not thick enough, were still standing at the present day. Sir Benjamin wrote : " If an engineer has not had some failures with retaining walls, it is merely evidence that his practice has not been sufficiently extensive ; . . ." His Paper should be read by every engineer, because, though written 56 years ago, it went almost as far as had now been reached. Engineers had been guided in the past by text-book theories of earth-pressure which took no account of many of the major factors, such as the all-important moisture-content. Only in recent years had the study of soil-mechanics been initiated, but even now the knowledge available was very limited.

In the case described by the Author experiments had been carried out at the Adelaide University on clay taken from the bores referred to on p. 127 and sealed in an airtight container. Curiously, however, the tests had not been carried out on the clay as received, which contained only 23 per cent. of moisture, but on clay containing 30 to 40 per cent. of moisture, the results being as given in Table VIII (p. 127). The calculated pressures which would be developed by clays con-taining such a high degree of moisture as 30 to 40 per cent. would be very high indeed, as shown in the Paper, but those developed by the clay in the dam, which contained only 23 per cent. of moisture, would be very much less. For that reason, Mr. Binnie considered that there must have been some other contributory cause for the subsidence ; that cause was to be found in the fact that the rockfill

[1] "The Actual Lateral Pressure of Earthwork." Minutes of Proceedings Inst. C.E., vol. lxv (1880–81, Part III), p. 140.

was founded on top of what was called " clayey material " in the paper and was described as " clay " in the Report of the Eildon Weir Inquiry Board, which, when moist, would form a very greasy surface on which the rockfill would slide under comparatively little lateral pressure.   There was evidence that the rockfill itself would stand at a very high angle of repose and would only have exerted slight lateral pressure, so that its movement could only be accounted for by the greasiness of the clay foundation.

On p. 132 the Author suggested that an economical form of rock-fill dam would comply with three requirements, of which the first was " A foundation of sound clean rock free from overburden."   To fulfil that requirement, however, would frequently involve prohibi-tive excavation.   Suitable initial precautions could, however, be taken even when dealing with a greasy, slippery soil ; for instance, slipping of the bank could be prevented by putting a very heavy toe to the bank and driving piles well down through that toe along the whole length of the bank.   Such a method would normally be adopted in circumstances of the kind in question.   In the present case it had been impossible to de-water the reservoir so that such a toe could be put in ; the object had had to be achieved by throwing extra weight on that portion of the dam.

When rock was dumped and no measures were taken to consoli-date it, the subsidences both vertically and horizontally were very considerable; in one case [1] subsidence had not ceased even after 50 years.   When it had been necessary to consider how to place the rock of the Shing Mun dam, which was 275 feet high above the stream-bed level, it was decided to pack the stones by hand so as to minimize the settlement due to consolidation of the fill.   The base of the fill was not taken down to the rock but to boulders and sand which appeared to be well compacted in the old bed of the stream.   There was a very narrow gorge with steeply-inclined sides rising to a height of about 100 feet, above which the valley widened out.   It was very difficult to distinguish the settlement due to compaction of rockfill from that due to the compression of the soil by the superincumbent mass.   To ascertain what settlements took place where the greatest stress was thrown on the foundations, three plates were built into the rockfill ; one on each side of the gorge close to the bottom, and one over the centre of the gorge and 140 feet above the bottom. Those plates having been built in, rods were attached to them and brought up with the rockfill as the work was carried on, records being kept of the lengths of rod added, so that at any time the distance between the top of each rod and the plate below it was

Mr. W. Binnie. known, and, the original level of the plates being known, it was
possible to tell whether there had been any depression.   The results
were very curious.   The two plates which had been put one on each
side of the gorge close to the bottom sank gradually at an average
rate of 0·022 foot per month for 20 months until the calculated
pressure on the foundations reached slightly less than 7 tons per
square foot.   Suddenly the material under the foundation appeared
to break down and the rate of settlement became 0·32 foot per month,
which went on for 10½ months, the total settlement when the work
was completed amounting to 3·75 feet at plate No. 1.   The rods then
had to be removed, because of the squared granite facing which had
to be put over the top of the rockfill, and while that was being placed
no levels could be taken.   As soon as the facework was in place
levels were again taken to see whether settlement continued ; during
the subsequent 7 or 8 months there had not been the slightest trace
of further settlement.   The next question was whether the move-
ment was entirely due to compaction of the underlying material or
whether there had been any settlement of the rockfill in itself.   That
question was answered by the settlement-record of the third plate,
which had been placed 101 feet above plate No. 1 ; simultaneous
readings during the succeeding 16 months showed that the fill
between the plates, 101 feet in vertical height, had not compacted
as the settlement of the two plates was the same.   It was evident,
therefore, that the hand-packed rockfill had not been appreciably
compressed.   At Shing Mun dam there was a very large retaining
wall which formed the water-face of the rockfill ; when the reservoir
was filled and overflowed to a depth of 2 feet the maximum down-
stream movement of that wall was only $\frac{7}{16}$ inch, which showed the
effectiveness of the support given by the rockfill.

For rendering the rockfill watertight the Author advocated
(p. 132) " an upstream reinforced-concrete slab of the ' articulated '
type supported directly on the fill and continued downward into the
bedrock as a cut-off, along the upstream toe, the depth of the cut-off
being determined by the nature of the rock foundation.   The con-
nexion between the slab and cut-off would be designed to allow of
settlement of the fill by forming a ' hinged ' watertight joint with
special provision for drainage."   That method of construction,
however, would involve a longer cut-off trench, as it would have to
traverse the whole periphery, and the cost of getting the cut-off
trench down to the rock often bore a very considerable proportion
of the total cost of the work.   Moreover, it was difficult to devise a
joint which would be always watertight while connected to a slab
which was bound to slide up and down on the top of the fill due to
changes of temperature and settlement.

Mr. EDWARD SANDEMAN extended his sympathy to the engineers Mr. Sandeman. who had been called upon to reconstruct the dam. In the circumstances that had been described, unless the whole central portion of the dam were pulled down there was a great risk that some weak portion in the corewall would not be discovered.

The failure of the Lower Otay dam, California, built in 1887–97, was interesting. That dam had been of similar type and dimensions to the Eildon dam; it had stood for about 18 years, but in January, 1916, during a flood, the water rose to the spillway-level and the dam was washed away. In that instance the rockfill had been mixed with earth. As the Author pointed out in the Paper, the design of the Eildon dam—as of the Lower Otay dam—was such that only the downstream half of the mass of the dam was acting in opposition to the pressure of the water and of upstream rockfill. For the safety of such a structure it was essential that the material on either side should be of a rigid type.

It had been found by experience that hand-packing of rockfill made a sounder structure and allowed a watertight face to be placed on the upstream slope of the dam, so that the watertight face had the whole mass of the dam behind it. That design had been developed by Italian engineers in particular, who had built such dams up to 300 feet high.

It would be interesting to know whether the removal of the central part of the Eildon embankment had been contemplated. He supposed that the undesirability of dewatering the reservoir had prevented that being done.

He would like to congratulate the engineers responsible on the extremely small amount of leakage—only 50,000 gallons per day—which occurred after the very difficult repairs had been carried out.

Mr. JAMES WILLIAMSON observed that it was much easier to Mr. criticize a difficult engineering work in retrospect than to arrange Williamson. and foresee everything that should be done at the beginning. Several important lessons might, however, be learned from the Paper.

From *Figs. 12* (p. 137) it would be seen that before the occurrence of the principal slip the corewall had already moved $4\frac{1}{2}$ feet downstream at chainage 1,550 and nearly 5 feet at chainage 2,300. It seemed clear, therefore, that considerable movement had taken place long before the big slip occurred, and that even without that slip there was probably a risk of some failure. The movement referred to also appeared to support Mr. Binnie's suggestion that settlement of the ground underneath the bank had taken place. In Figs. 9, Plate 1, on the bore shown farthest to the right there was a mark " N.S." at a depth of $2\frac{1}{2}$ or 3 feet below the original surface-line. Assuming that " N.S." meant " natural surface,"

Mr.
Williamson.
there was evidence that at that particular bore the natural surface
had gone down 2½ or 3 feet. It was quite likely that if the natural
surface had settled on the reservoir side it had also settled on the
downstream side, as indicated in *Fig. 27*.

With regard to the deflexions downstream of the corewall, it
was most remarkable to find from *Figs. 11* (p. 129) that there was
no downstream deflexion at all at the two places where the corewall
had a radius and was in the form of a horizontal arch. He had
made some rough calculations, and had found that those two por-
tions, even though they were only 3 feet thick, would be capable
of holding up—though at a fairly high concrete-stress—a very large
part of the actual pressure from the upstream side. The curved
portions would act as arches and the straight portions as abut-

Fig. 27.
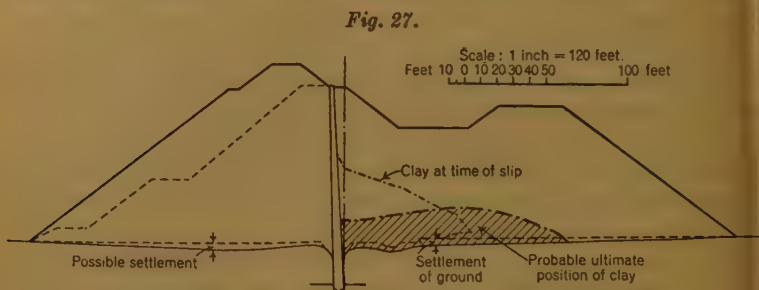
ments, and the corewall, though very thin, would not be liable to
buckle as it was held on both sides. That, he thought, completely
explained why the dam had not yielded at the two circular portions
of the corewall, and it indicated that the conditions producing the
downstream movement had been marginal so that the extra support
provided by those curved portions had been sufficient to stop all
downstream movement in their vicinity. It appeared that if a little
more care had been taken to put rather a wider bank on the down-
stream side, with rather a flatter slope, and if it had been better
built and not merely tipped to the angle of repose, the movement
downstream might not have occurred.

The trouble on the upstream side had evidently been due entirely
to the clay and to the extra pressure that it introduced. *Fig. 27*
showed the probable ultimate position. Fig. 9, Plate 1, was possibly
a little misleading in that the position of the clay at chainage 1,550
was shown at the time of the major slip, the top of the rockfill
having subsided about 25 feet. *Figs. 12* (p. 137), however, made it
clear that before the rockfill came finally to rest it went down
another 50 feet, so that the final position of the clay was not as

shown in Fig. 9, Plate 1, but might be more or less as shown in *Fig. 27*. Clay which would yield to that extent would be capable of exerting a fluid pressure far greater than that of water ; moreover, as it was plastic and partially fluid, pieces of rock would sink into it and would form a mixture just like soft concrete, which, owing to the stone being denser than the clay, would exert even larger pressures than the clay alone. It appeared to him, therefore, that if in the first place the engineers had relied on the concrete corewall and had put rockfill alone on the upstream side, there might not have been the downstream movement and there certainly would not have been the slip.

With reference to the remedial measures, it seemed to him that the arrangement which had been adopted on the upstream side was very well calculated to effect the ultimate purpose, although a great deal of rock had had to be tipped before stability could be attained. On the downstream side, he doubted whether it had been very wise to pile up the weight of rockfill on a narrow base in the way shown. It would probably have been better to have gone wider and to have finished the fill with a flatter slope.

The wedge-like effect of the pressure of the clay was well illustrated by the description of the movement in the vicinity of the outlet-tower. The clay was between the wall and the tower : the wall was pressed downstream, and the tower was pressed upstream.

The difficulty that had been experienced in operating the upstream gate-valves might have been due partly to an effect which was sometimes neglected. The valves were operated by rods from a double-acting hydraulic cylinder. When a valve of the shape employed (whether of box form or finished with a rib at the bottom) was cracked open under high pressure there was a tendency to complete release of pressure and even to the formation of a vacuum underneath the bottom of the gate. The force required to raise the gate was thus increased by the weight of water on the top of the gate. Unless the pressure downstream were relieved by adequate ventilation, there would also be a vacuum behind the gate which put an additional horizontal load on the gate and thus increased frictional resistance. In the case of high heads of water, those effects could increase the load to be lifted by 100 per cent., and he would like to know whether they had been partly responsible for the difficulty met with in operating the Eildon gate-valves. A modified design was now being more commonly adopted for high heads, the position of the valve being shifted from the downstream side of the shaft to the upstream side, and the thrust on the gate being taken by extending it into grooves. When a valve of that design was cracked open

<div style="float:left">Mr.
Williamson.</div>

there was no water-pressure on the top of the gate and no vacuum below or downstream of the gate ; moreover, when the gate was shut there was complete access by means of the shaft to the outlet-conduit.

In considering whether there was any weak point still in the dam, he thought that reference might be made to chainage 2,300, where there was still a mass of clay behind the corewall almost to the full height, and where the deflexion of the corewall had been over 7 feet. Perhaps, also, the authorities in charge of the dam might feel more comfortable if a separate outlet-tunnel of large capacity were driven through the rock at the side of the river to supplement the present outlet-culvert.

<div style="float:left">Mr. Cooling.</div>

Mr. L. F. COOLING, of the Building Research Station, suggested that the subsidence of the upstream bank might be accounted for by a theory different from that put forward in the Paper. According to that alternative theory, failure was the result of a shear slide, basically of cylindrical form, passing through the clay core and through the layer of clay in the foundation of the embankment. As was well known, failures in clay slopes usually took the form of shear slides, and their mechanism had been analysed by W. Fellenius,[1] H. Krey,[2] K. Terzaghi [3] and others. It seemed to him that most of the phenomena observed at Eildon could be explained on the basis of a shear slide, and by using the Fellenius-Krey method it was possible to make a rough analysis of the stability of the bank.

Fig. 28 was essentially a reproduction of Figs. 9, Plate 1, and showed the original and final sections of the bank and corewall. The surface along which the slip appeared to have occurred was shown by the line ABC, and the dotted lines indicated an intermediate position of the bank and corewall. The arc ABC was part of a circle with centre O, and it was suggested that the wedge of earth ABCDA rotated about that centre. In doing so, the clay core would tend to move away from the concrete diaphragm, but, as it would not be able to support the weight of the rockfill, it would deform and slide down the wall. Similarly the upstream toe, although tending to rise, would not be able to support the weight of the fill, but would deform and slide on the natural surface beyond the toe. As a result of that movement, the clay core would assume a shape not unlike that shown in Fig. 9, Plate 1, and in fact the curvature at the top and the pointed portion at the bottom seemed to lend support to the idea of the shear slide.

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[1] "Erdstatische Berechnungen," Berlin, 1927.
[2] " Erddruck Erdwiderstand," Berlin, 1936.
[3] *Public Roads*, vol. 10, No. 10, December, 1929.

Mr. Cooling.

In describing the subsidence, the Author drew attention to four important facts :—(i) The movement of the bank took place over that portion of the dam supported not on bedrock, but on a layer of clay. The arc ABC in *Fig. 28* gave a depth of the slip-surface of 10 feet below the natural level, and it appeared from the measurements given in Figs. 3, Plate 1, that the clay under the bank was probably between 10 and 15 feet thick. (ii) The outward movement of the upstream toe increased progressively towards the central chainage, and was in direct proportion to the downward movement of the clay ; that rather suggested a rotational movement of a mass as a whole. (iii) The clay core appeared to move as a mass and was not merely forced into the interstices of the fill ; that was

Fig. 28.
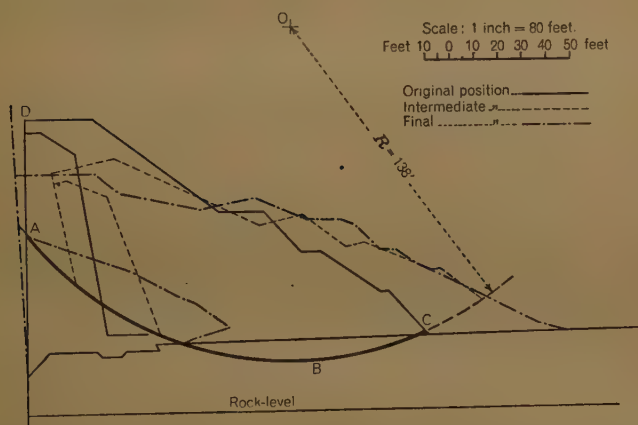
consistent with the idea of a shear slide. (iv) The failure occurred at a time when the reservoir-level had been drawn down to R.L. 776 for the first time after reaching full-supply level. The last-mentioned point was probably the most important of all, as would be appreciated from a rough analysis of the stability of the bank.

The principle of Fellenius's method of analysis was to consider the stability of the mass ABCDA (*Fig. 29*) against rotation about the centre O. The forces tending to cause rotation were derived from the weight of the mass, and the forces resisting the movement were derived from the cohesion of the clay and the frictional forces around the arc ABC. Those forces could be estimated from Coulomb's law of shear, which stated that the resistance along a shear surface could be represented by the equation $t = k + n \tan \phi$, where t denoted the shear resistance along the surface, ϕ the effective angle of internal friction, and n the normal pressure on the plane of

Mr. Cooling.

shear. The mass ABCDA was divided into a number of vertical slices, and the weight of each portion was calculated. The weight was then resolved into two components, one tangential to the curve, T, and one normal to it, N. In considering the stability of such a mass, if the forces between the slices could be neglected then the moment causing rotation might be expressed as $R\Sigma T$, the sum of the tangential components of the weights of the slices multiplied by the moment-arm of the rotation. The resisting moment was made up of two components, $R\Sigma N \tan\phi$ due to friction and $R\Sigma kl$ due to cohesion (where l denoted the length of the arc at the base of the slice). When failure occurred, $\Sigma T = \Sigma N \tan\phi + \Sigma kl$.

Fig. 29.
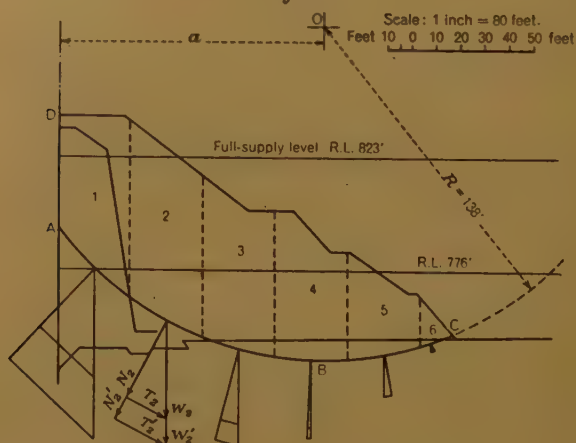
In applying that type of analysis to the failure under consideration, he had only been able to obtain a general idea of the stability, because it had been necessary to make various assumptions. Firstly, it was assumed that the clay in the clay core and in the foundation-layer had the same shear characteristics. That made the calculation much simpler, and was a reasonable assumption, as it was stated in the Paper that the clay core was actually made from the foundation clay near the site. Secondly, the resistance to movement down the concrete diaphragm was taken as being due to the cohesion of the clay, so that the resisting moment was equal to (cohesion × length of wall DA × moment-arm a). Analyses had been made for two conditions (*Fig. 29*), firstly with the reservoir full to R.L. 823, and secondly with the water drawn down to R.L. 776.

In the first case the weight of the rockfill in water was taken as 56 lb. per cubic foot; that figure had been calculated from the

Mr. Cooling.

Author's statement that the fill weighed 90 lb. per cubic foot in air, on the assumption that the rock had a specific gravity of 2·65. The weight of the clay core in water had been taken as 61 lb. per cubic foot, since in air it was stated to be 123 lb. per cubic foot. The weight of each of the blocks 1 to 6 (*Fig. 29*) was then calculated, and resolved into components normal to the shear surface (such as N_2) and tangential to the shear surface (such as T_2). The total disturbing moment $R\Sigma T$ was then computed. The resisting moment due to friction, $R\Sigma N \tan\phi$, was next calculated, taking $\phi = 5$ degrees as given in the Paper. The difference between those moments gave the value of the moment (cohesion along DA $\times a$) + (cohesion along ABC $\times R$) that was necessary to maintain equilibrium, whence the cohesion of the clay worked out to 625 lb. per square foot.

In the second case, when the reservoir was drawn down, there would be no hydrostatic uplift above R.L. 776 level, and the clay core would assume a weight of 123 lb. per cubic foot and the rockfill a weight of 90 lb. per cubic foot. The calculation was exactly similar to that for the first case (the forces being greater, as shown by W_2', N_2' and T_2', *Fig. 29*), and showed that when the reservoir was drawn down the cohesive strength of the clay would have to be as great as 950 lb. per square foot to ensure equilibrium.

For clay that had been remoulded at a moisture-content of 30 per cent. of the dry weight, Table VIII (p. 127) gave the friction angle as 5° 10′ and the cohesive strength as 820 lb. per square foot. The samples taken from the clay core were of stiffer consistency, with a moisture-content of 23 per cent., so that the cohesive strength of the core was probably reasonably near to the value deduced above from the analysis of the stability of the bank in the second case. It seemed, then, that that rough analysis, made on the basis that the failure was a shear slide in the clay, gave results that agreed reasonably well with the data for the clay.

A further point which might be considered was the movement of the bank during reconstruction. With a mechanism of failure such as that indicated, it was evident that to prevent appreciable further movement it would have been necessary to place considerably more rockfill on the upstream berm than at the corewall. As was shown in *Figs. 12*, p. 137, however, similar quantities had been placed at the corewall and at the toe between May and November 1929 ; movement had continued during that period, and had not been arrested until the loading of the berm preponderated.

In research into soil problems, progress was bound to depend largely on information derived from the study of the behaviour of full-scale structures, and records of observations and measurements

Mr. Cooling.

such as were included in the Paper were of great value. The utility of such records would be enhanced if they included the results of tests carried out on " undisturbed " samples from the various strata.

Mr. G. Binnie.

Mr. G. M. BINNIE observed that the Author, referring to rockfill dams with an upstream reinforced-concrete slab of the articulated type, stated that with the upstream rockfill placed as a well-graded mass, a uniform bearing for the slab could be prepared, and comparatively even settlement would be assured. The San Gabriel No. 2 dam, 265 feet high, had been designed on that principle. According to a published account,[1] it was constructed of loose rock of excellent quality in 25-foot lifts, a few inches of fine material being placed at the top of each lift to facilitate truck-hauling. The settlement of the rockfill during construction was stated to have been about 1 foot per lift. Both the upstream and downstream slopes were approximately at the natural angle of repose of the rock, the upstream slope being constructed of dry rubble 6 to 15 feet thick. The facing was constructed in laminated concrete slabs. About 150,000 cubic yards of overburden were removed from the site before any rock was placed. After heavy rain which fell when the dam was nearing completion, a vertical settlement of about 12 feet occurred, and the laminated concrete face was crushed and cracked to such an extent that it had to be entirely replaced by a temporary timber facing. Settlement and horizontal movement of a rockfill dam might be due to any of the following causes :—(i) the compression of the foundations under load ; (ii) the crushing of the bearing points of the rock under increased load ; (iii) the displacement and readjustment in position of the stones under load, the smaller stones tending to move into the voids between the larger stones ; and (iv) the washing-out by rain of the fine material between the bearing points of the stones. In view of the large amount of overburden removed and the rocky nature of the site, it hardly seemed probable that the sudden settlement at the San Gabriel No. 2 dam was caused by the foundations becoming soft under the influence of rain. It was also evident that rain itself would not affect either the crushing of the bearing points of the rock or the equilibrium of the stones themselves, provided they were all bearing directly upon each other " rock to rock." What appeared to have happened was that the rain washed the fine material from between the bearing points of the stones into the voids. As a result, the equilibrium of the stones themselves was disturbed and a general readjustment occurred, resulting in settlement which continued until a sufficient number of stones were bearing on each

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[1] *Engineering News-Record*, March 7th, 1935.

other " rock to rock " to re-establish equilibrium. The Author Mr. G. Binnie. suggested on p. 133 that consolidation of rockfill could be attained by tipping fines amongst the larger stones, but, judging from the behaviour of the San Gabriel No. 2 dam, that would seem to be a dangerous practice unless the fines were sluiced with water into the voids.

It was interesting to compare with the San Gabriel No. 2 and Eildon dams the construction and settlement of the rockfill, 275 feet high, of the Shing Mun dam recently completed in Hong Kong. The foundations of the rockfill consisted of badly decomposed soft granite and boulders on one bank and rock on the other, only the top soil having been removed. The rockfill was constructed of stones ranging in weight from 4 tons to a few pounds, the larger stones being set by derrick and the rest by hand, in such a manner that the interstices between the stones were kept as small as possible Quarry-waste and fines were eliminated as far as possible from the fill, which was constructed in 2-foot layers to enable easy inspection for bad packing, and each layer had a slope of 1 in 10 upwards in the downstream direction. The settlement of the rockfill during and after construction had already been mentioned by Mr. W. J. E. Binnie (p. 194) ; it was remarkably small, and the first torrential rains at the beginning of each wet season had had no effect whatever upon it. That very satisfactory result could undoubtedly be attributed to the hand-packing and the avoidance of fines in the rockfill.

The cost of placing the hand-packed rockfill after it had been dumped by derrick on the dam was approximately 7d. per cubic yard. Where labour was cheap—the maximum rate for unskilled labour for the Shing Mun dam was $1\frac{1}{2}d$. per hour—there was no doubt that that was the ideal form of construction.

Mr. W. T. HALCROW agreed with the Author's remark on p. 139 Mr. Halcrow. that the thickness of corewalls required for rockfill dams was largely a matter of judgement and experience. The thickness of the corewall of the Eildon dam ranged from 2 feet at the top to 6 feet at 137 feet down, whereas the corewall of the Treig dam,[1] which was partly rockfill, was 5 feet thick at the top and 10 feet thick at 40 feet down, though it was a much smaller dam. The trouble experienced at Eildon did not appear to have arisen from the corewall, although it seemed to be thin considering the different nature of the materials on either side and the fact that the rock was not consolidated in any way. In the case of Treig dam the rockfill had been rolled in

[1] A. H. Naylor, " The Second-Stage Development of the Lochaber Water-Power Scheme." Journal Inst. C.E., vol. v (1936–37), p. 3. (February, 1937.)

Mr. Halcrow.   layers of about 2 feet in depth by a 10-ton roller, and it had been found that by keeping the downstream bank some 8 to 10 feet higher than that upstream during construction the massive reinforced concrete corewall could be deflected in an upstream direction. Mr. Halcrow had not previously seen rockfill rolled. When an embankment was formed of stone of irregular shape, many pieces rested on points which crushed as load was built up and caused settlement. It seemed reasonable to anticipate that crushing by artificial consolidation, and good results had been obtained. The articulated reinforced-concrete slab resting on the rockfill and forming part of the spillway had not settled since the dam was completed about 4 years ago.

Whilst there was something to be said for placing special material upstream of the corewall to stop seepage through cracks in the concrete, Mr. Halcrow doubted if it was of much value. Rockfill downstream formed a natural drainage-medium, and the percolation of a small amount of water was not likely to be serious. For the Treig dam no material of a clayey nature could be found in the district to place against the corewall, and therefore dust rejected from the stone-crushing installations had been used. If any cracks occurred in the corewall that fine stone-dust would be drawn into them by the water.

The failure of the bank at Eildon reservoir was instructive, and the Paper was of great value. It was easy to be wise after the event, but the results indicated that material having a weight approaching that of the clay would have been more suitable than loose rock to hold it in position; perhaps if the rock had been consolidated by rolling, the increased weight might have been sufficient to ensure stability. Probably no trouble would have been experienced if the clay had been omitted.

Reference had been made in the Paper to the type of dam which was formed of a rockfill embankment with an articulated impervious slab on the upstream slope. He had on two occasions prepared detailed designs for a dam of that type, and had rejected both of them for various reasons. Some of those reasons had been mentioned in the Discussion, but another important one was the difficulty of making a satisfactory joint between the vertical cut-off wall at the toe of the upstream bank and the sloping slabs on the rockfill where following up the steep sides of a valley. The central core had many advantages, and, if made thick enough to support the downstream bank when the reservoir is empty, it could be used without a bank upstream or with a partial bank as in the case of one section of the Ericht dam of the Grampian water-power scheme.

The works described in the Paper showed that the design of

ams was a long way from becoming the subject of a British Standard Mr. Halcrow.
Specification.

Dr. W. L. LOWE-BROWN mentioned that Professor C. F. Jenkin, Dr. Lowe-Brown.
referring to the Eildon dam,[1] had explained its failure as being
due to lack of shear strength of the foundation.

*** Mr. A. L. BELL observed that Papers such as the present one Mr. Bell.
which gave intelligent descriptions of failures were perhaps the most
valuable of all.

He thought it desirable to point out that the formula (p. 126) for
the intensity of horizontal pressure at any depth h,

$$p = wh \, \tan^2\left(\frac{\pi}{4} - \frac{\alpha}{2}\right) - 2 \, k \tan\left(\frac{\pi}{4} - \frac{\alpha}{2}\right),$$

had been devised to give the pressure exerted by an indefinitely large
area of clay, that being the condition prevailing at Rosyth. How
far that formula could be trusted to give the pressure arising in a
wall of clay ranging in thickness from 27 to 37 feet he had not con-
sidered. The value of the angle α also depended upon the scales used
on the horizontal and vertical axes of the diagram upon which
the results of shearing tests were plotted. In his own diagrams he
had used identical scales of tons per square foot for both axes. The
Author, he noted from *Fig. 10* (p. 126), had used unequal scales for
those axes.

When what he had termed the critical depth in a mass of clay was
reached, the clay began to flow. He did not believe that the lateral
pressure exerted by clay ever equalled that of a liquid of equal weight
per cubic foot, though under very great pressure there would doubt-
less be a close approximation to that condition. A true fluid was
incapable of offering any permanent resistance to internal shear, but,
in his view, clay never reached that condition. There was in it, he
thought, a capacity for resistance to shear which, under pressure,
though it might be overborne, was not wholly eliminated. If that
were true it followed that the lateral pressure of clay would always,
to some extent, fall short of that exerted by a true liquid of equal
density.

He had once had an opportunity of observing, on works in progress,
how very slowly clay flowed when the critical depth had been slightly
exceeded. A monolith had been sunk to a depth of about 85 feet
below Ordnance datum. The cutting edge on one side had encoun-

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*** This and the following contributions were submitted in
writing.—SEC. INST. C.E.

[1] *Discussion on* "The Pressure on Retaining-Walls." Minutes of Pro-
ceedings Inst. C.E., vol. 234 (1931–32, Part 2), pp. 143 and 174.

Mr. Bell.

tered, and been sunk into, rock. Further sinking seemed unnecessary, especially as the greater part of the foundation exposed at the monolith-bottom was hard boulder-clay with stones. In one corner, however, there was a relatively small area of clay of a different character and without stones. It was firm enough to support a man walking over it, but was not as hard as boulder-clay. He gave instructions for that small patch to be dug out in order to expose the boulder-clay which, it was presumed, lay below it. That was done to a depth of several feet without reaching boulder-clay. To his surprise he found that, on resuming work next day (or after a week-end) the excavation in this small area had filled up again. After several attempts, each giving the same result, the effort to reach a firmer bottom was abandoned. The rate of flow was so slow that it could not be perceived by the eye. The intensity of the minimum downward pressure required to stop that upward flow was, according to the theory advanced by him,

$$p = wH \tan^4\left(\frac{\pi}{4} - \frac{\alpha}{2}\right) - 2k \tan^3\left(\frac{\pi}{4} - \frac{\alpha}{2}\right) - 2k \tan\left(\frac{\pi}{4} - \frac{\alpha}{2}\right)$$

He noted that the failure at Eildon was attributed to the sliding, upstream, of the clay along its base. That might be the case, but there was the possibility that the initial failure had taken place in the virgin material upon which the dam and clay bank had been founded.

The decision to secure impermeability by heaping clay against the upstream side of the corewall was perhaps natural, though unfortunate as matters turned out. It was, of course, easy to be wise after the event. Perhaps a better plan in future cases would be to use clay imprisoned between two parallel corewalls sufficiently bonded together.

Mr. Evershed.

Mr. W. A. EVERSHED observed that the subsidence was somewhat similar to a slip towards the water which he understood had occurred at Waghad dam, an earthen dam near Nasik, Bombay Presidency, about 1915. The soil concerned was black cotton soil or a mixture of black soil and *murram*, and the slip was caused by its own superincumbent weight. Black soil was largely clay, and was never allowed to be used in earthen dams or canal-banks for the purpose of watertightness without an admixture of *murram* (the proportions being ascertained by experiment) owing to the shrinkage and cracking of black cotton soil when alternatively wet and dry. Again, in subsoil-drainage works in the irrigated area of the Deccan, clayey subsoils had been found in places to have become deflocculated in the presence of excess water, especially where sodium salts predominated, the liquid mud so formed causing

reat difficulty in the excavation of open trenches and the laying Mr. Evershed. f pipe drains. Research in the soil-physical laboratories of the pecial Irrigation Division indicated that the most satisfactory nethod of dealing with the mud was to flocculate it by forcing a olution of calcium chloride into the soil round about the drain to e made. It appeared, therefore, that where soil or clay was to mdergo stress in direct contact with water—especially where alterating with a dry state—the material to be used should be subjected o analysis and research by an expert in soil-physics in collaboration vith the engineers.

It would be interesting to know whether any models of the Eildon pillway and gate-openings had been made to ascertain the disharge-formulas, and, if so, how the results obtained compared vith those given in the Paper. In his opinion it was only possible o gauge large discharges accurately either after experiments with models to ascertain the discharge-formulas, or by means of speciallyouilt measuring devices such as standing-wave flumes, carefully lesigned for the particular circumstances. In the Special Irrigation Division at Poona, India, a model of the sluice-gates of a large nasonry dam impounding irrigation-water was made, and the esults differed considerably from those given by the theoretical ormula previously used.

Sir HENRY JAPP observed that the slip appeared to be due to Sir Henry hear failure in the foundation-clay. Such failures were attracting Japp. nore and more attention, and had been studied in great detail by Dr. K. Terzaghi, who had based his investigation on observations of actual slides of banks of cohesive earth. He had pointed out that such slides never took place along planes but always along cylindrical surfaces. There had been several cases in England ecently, which were being investigated by Dr. R. E. Stradling and nis staff at the Building Research Station, Watford. It might be questioned why the Eildon bank did not slip until the level of the vater was greatly reduced. According to the above theory it eemed that slips of that nature occurred when the shear-strength of the clay, which varied inversely with its moisture-content, was no longer able to bear the weight of the bank. The 15-foot bed of lay on which the Eildon reservoir bank rested had probably very ittle moisture-content while the trench for the corewall was being unk. After the reservoir was filled, however, that clay would commence taking up moisture very slowly from the overlying water. At some period, as the moisture-content increased, the shear-strength vould be reduced to a point where the factor of safety became very mall. When the water-level was then lowered by 46·5 feet the lay and rubble bank remaining above water-level increased in

Sir Henry
Japp.

weight, and the increased load combined with the reduced shear-strength resulted in the clay under the bank giving way along a cylindrical surface of cleavage.

Mr. Lloyd.

Mr. H. G. LLOYD observed that he understood that the price of cement at the Eildon works was 7s. per bag. In view of the fact that the 112-lb. bag had not yet been fully accepted in England, it would be useful to know what the weight of cement was in the bags used.

.*. The Author's reply to the Discussion, together with the Correspondence, will be published in the Institution Journal for October, 1938.—SEC. INST. C.E.

SUBSIDENCE OF A ROCKFILL DAM AT EILDON RESERVOIR, AUSTRALIA.

PLATE 1.
SUBSIDENCE OF A ROCKFILL DAM AT EILDON RESERVOIR, AUSTRALIA.

Figs: 3.

PLAN.

LONGITUDINAL SECTION A B.

PLAN AND SECTION OF COMPLETED DAM.

Figs: 5.

AS CONSTRUCTED (1927).
CHAINAGE 950 FEET.

AFTER SUBSIDENCE (1929).

AS RECONSTRUCTED (1935).

AS CONSTRUCTED (1927).
CHAINAGE 1,550 FEET.

AFTER SUBSIDENCE (1929).

AS RECONSTRUCTED (1935).

AS CONSTRUCTED (1927).
CHAINAGE 2,300 FEET.

AFTER SUBSIDENCE (1929).

AS RECONSTRUCTED.

CROSS SECTIONS OF DAM.

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POSITION OF CLAY AT VARIOUS CHAINAGES.

SECTIONS AT CHAINAGE 1,550 FEET, SHOWING BORES, CLAY, AND BANK AS RE-BUILT IN 1932.

EJECTOR-PIT.

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SHAFT AT 1,950 FEET.

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LONGITUDINAL SECTION ALONG MAIN OUTFALL-TUNNEL AT CHAINAGE 1,550 FEET.

BOTTOM PLAN.

DRAINAGE-SYSTEM.

CROSS-SECTION OF COREWALL-DRAIN TUNNEL.

The Institution of Civil Engineers. Journal. March, 1938.

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SUBSIDENCE OF A ROCKFILL DAM AT EILDON RESERVOIR, AUSTRALIA.

SUBSIDENCE OF A ROCKFILL DAM AT EILDON RESERVOIR, AUSTRALIA.

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AT COREWALL.

UPSTREAM OF H.O. TOWER.

DOWNSTREAM OF COREWALL.
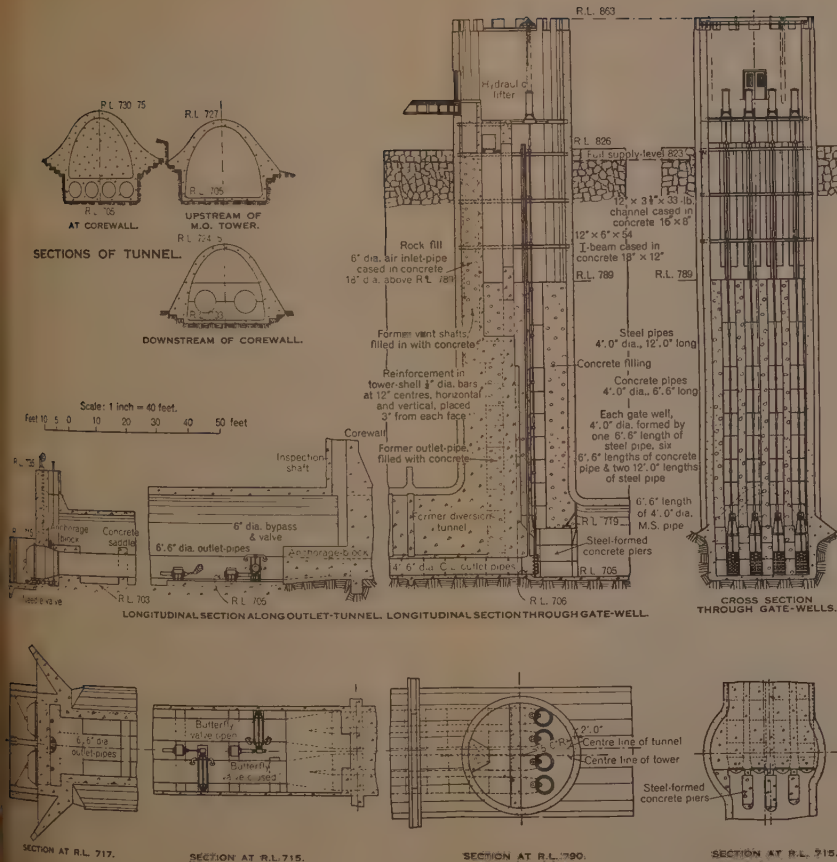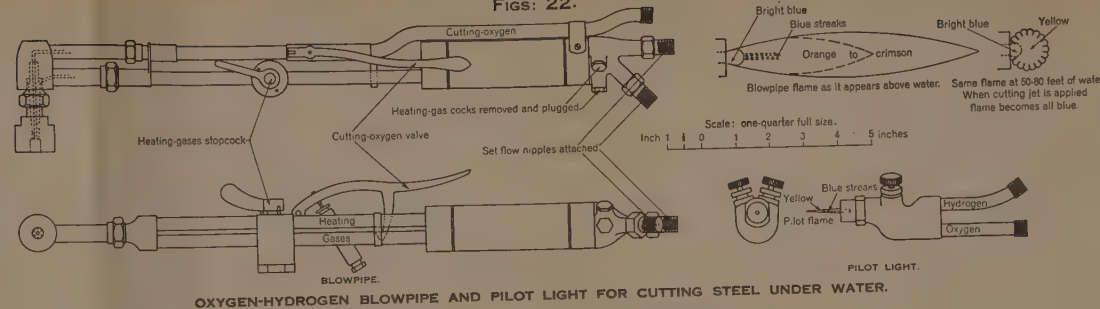
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SECTION AT R.L. 715.

SECTION AT R.L. 790.

SECTION AT R.L. 715.

MAIN OUTLET-WORKS.
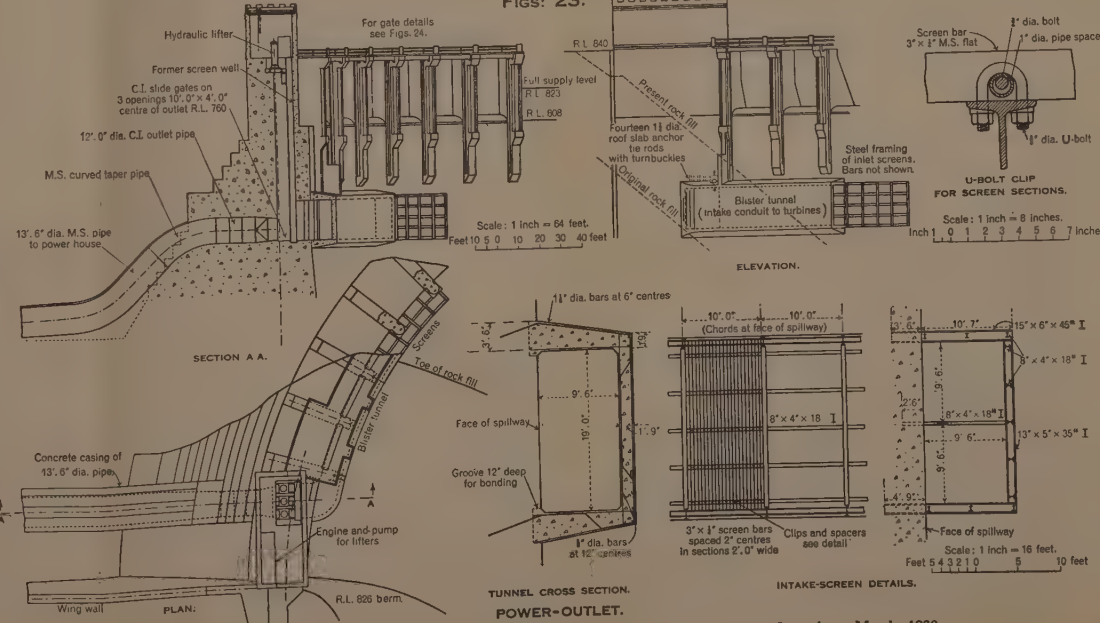
TUNNEL CROSS SECTION.

POWER-OUTLET.

PLAN.

INTAKE-SCREEN DETAILS.

RIVER

COULBURN

Power-house

PLAN.

SPILLWAY.

SECTION A.A.
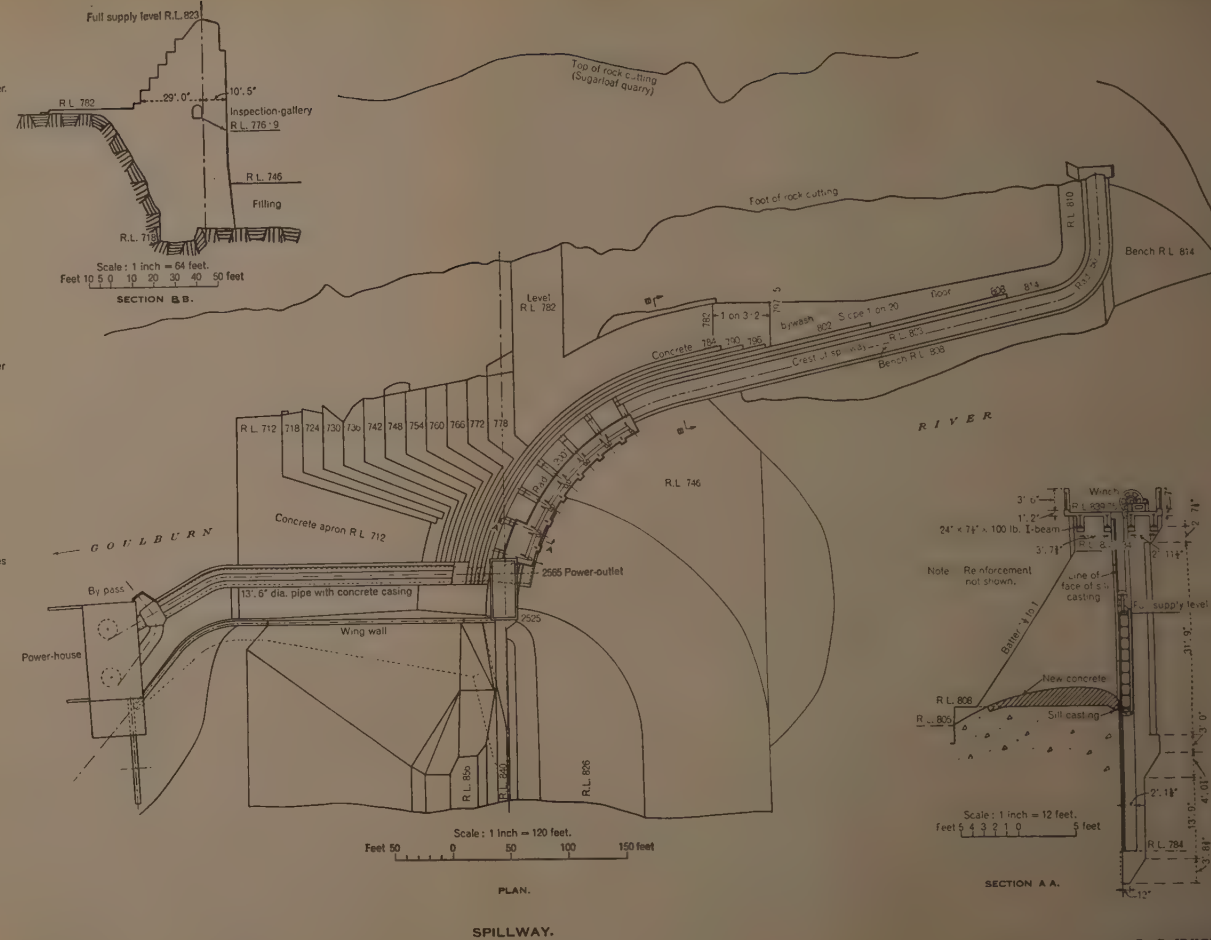
ORDINARY MEETING.

8 February, 1938.

SYDNEY BRYAN DONKIN, President, in the Chair.

The Scrutineers reported that the following had been duly elected as—

Associate Members.

RICHARD RAYMOND CARLTON.

JOHN WALTER CHIBBETT, Stud. Inst. C.E.

PETER LOVELL CUBITT, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.

STANLEY HERBERT DAINTY, B.Sc. (*Birmingham*), Stud. Inst. C.E.

HAROLD BISSET DUFF, B.Sc. (*Aberdeen*), Stud. Inst. C.E.

PERCY RICHARD DUNCAN, M.A. (*Oxon*), B.Sc. (*Birmingham*).

WILFRID FRANK GEORGE, Stud. Inst. C.E.

RICHARD HERMON, B.Sc. (*Edin.*), Stud. Inst. C.E.

ARTHUR WILLIAM HILL, B.Sc. (Eng.) (*Lond.*), Stud. Inst. C.E.

WILLIAM JOHN BLOIS JOHNSON, B.Sc. (Eng.) (*Lond.*).

SYDNEY KENNETH JORDAN, Stud. Inst. C.E.

EDWIN HAROLD JULIAN.

JOHN McNEE McGAW, Stud. Inst. C.E.

PETER MARTIN, Stud. Inst. C.E.

GEORGE JAMIESON MILLS, M.Eng. (*Liverpool*), Stud. Inst. C.E.

KAMAL KUMAR MITRA, B.Sc. (Eng.) (*Lond.*), B.Sc. (*Calcutta*), Stud. Inst. C.E.

GEORGE WILLIAM MORLEY, M.A. (*Cantab.*), Stud. Inst. C.E.

ERNEST PATRICK CHRISTOPHER NEILL, B.Sc. (*Belfast*).

JOHN AMBROSE PICKEN, Stud. Inst. C.E.

JOHN FREDERICK LIHOU ROBERTON, B.Sc. (*Lond.*).

TERENCE ROBERT BEAUMONT SANDERS, M.A. (*Cantab.*).

KENNETH MYERS SCOTT, Stud. Inst. C.E.

GEORGE CARNEGIE SMITH, B.Sc. (*Aberdeen*), Stud. Inst. C.E.

JOHN ARTHUR FRANK SMITH.

JOHN ROBERTSON STEWART, B.Sc. (*Witwatersrand*), Stud. Inst. C.E.

Associate.

ALLEN CHARLES LEWIS.

The following Paper was presented for discussion, and, on the motion of the President, the thanks of The Institution were accorded to the Author.

Paper No. 5110.

"An Experimental Investigation of the Effect of Bridge-Piers and Other Obstructions on the Tidal Levels in an Estuary."

By Professor ARNOLD HARTLEY GIBSON, D.Sc., LL.D., M. Inst. C.E.

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INTRODUCTION.

ANY proposal to erect bridge-piers, or works in any way restricting the waterway of a tidal estuary, is usually opposed by port authorities interested in the navigation of the estuary, not only on the grounds that such obstructions will increase the difficulties of navigation, but also because they will affect the flow of water into and out of the upper estuary. It has been generally considered that any constriction in the waterway must reduce the volume of water entering and leaving the upper estuary, and all authorities are agreed that any very appreciable reduction in the volume of flow would be likely to affect adversely the navigable channels up- and downstream. In the past no data have been available as to the probable magnitude of the effect of such obstructions in any particular case, and in view of proposals in recent years to build bridges across certain estuaries in Great Britain, the Author thought it might be of interest to attempt to measure the effect of obstructions of different magnitudes in a model of one of the estuaries concerned.

PREVIOUS TESTS.

Experiments on a Model of the Severn Estuary.

Some few years ago a model of the Severn estuary, to a horizontal scale of 1 : 8,500 and a vertical scale of 1 : 200, was constructed and

operated for the Severn Barrage Committee in the engineering laboratory of Manchester University. In the course of the experiments the effect of various designs of barrage was investigated. These were founded on the English Stones, a rock plateau about 5 miles upstream from Avonmouth. In each design water, on the flood tide, passes into the estuary between a series of sluice-gate piers, and afterwards passes between the piers of a rail-and-road viaduct. The obstruction of the waterway at high water varied in the different schemes from 63·6 per cent. of the cross section as a minimum to 82·8 per cent. as a maximum.

The effect of these obstructions on the level of high-water spring tides in the upper estuary at Beachley, immediately above the barrage, and at the upper extremity of the estuary, at Gloucester, was observed in the course of the investigation. After making slight adjustments for the differences of low water in the tidal basin in the different experiments, the figures are as follows :—

Obstruction : per cent. of cross section of waterway.	Lowering of high-water spring-tide level : inches.	
	Beachley.	Gloucester.
63·6	8·4	16·0
66·8	10·8	22·8
74·6	14·4	33·6
82·8	31·0	44·4

On plotting these figures on a base representing the percentage obstruction of waterway, it appears that the effect falls off very rapidly as the obstruction is diminished ; owing, however, to the fact that the smallest of these obstructions is so large, the extrapolation of this curve to give values for small obstructions is too speculative.

Experiments on a Model of the Dee Estuary.

In 1931 a model incorporating the estuaries of the Dee and Mersey and a considerable area of Liverpool Bay was constructed, and was operated in the engineering laboratory of Manchester University. This had a horizontal-scale ratio of 1 : 7,040, and a vertical-scale ratio of 1 : 192.

In the course of the investigation, the effect of a suggested embankment and bridge across the mouth of the estuary of the Dee between Point of Air and Hilbre Point was examined. The line of the proposed work is shown in *Fig. 1* (p. 212). It was to consist of two solid embankments connected by a central bridge having thirteen spans, with piers each 28 feet wide at 330-foot centres. The width of the

estuary at this site is 5·25 miles and the area of the cross section at high-water spring tide is approximately 950,000 square feet. The width of the waterway between the proposed piers is 3,940 feet, and the obstruction of the cross-sectional area of the estuary at high water caused by the work is 74 per cent.

Tide-curves were taken in the model at points corresponding to Llanerchymor, Heswall, Pentre, and Connah's Quay, before and after the introduction of the bridge and embankment, the tidal range at spring tides in each case being 31·9 feet at Liverpool. The curves taken immediately after the introduction of the obstruction, before any substantial change had taken place in the channels of the upper estuary, showed that the mean level of high water had been lowered ·by 1·2 foot and the mean level of low water had been raised 0·6 foot.

Fig. 1.

ESTUARY OF RIVER DEE.
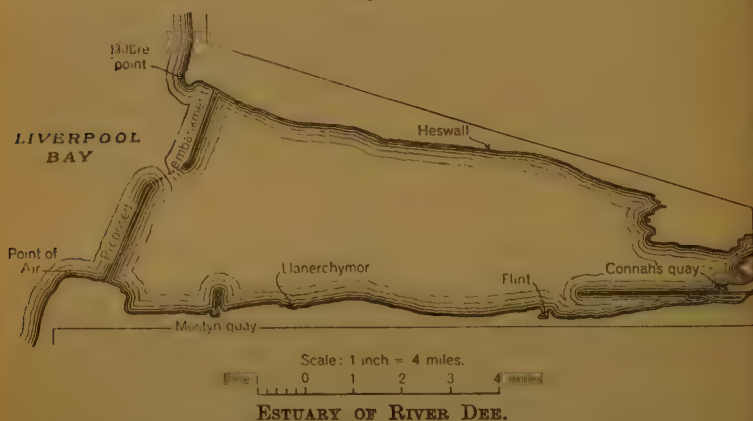
This lowering of the mean high-water level compares with the figure of approximately 2·0 feet in the Severn with the same percentage obstruction. Since the spring-tide range at the obstruction is 29 feet in the Dee and 41 feet in the Severn, the ratio of the changes in high-water level is of the same order as that of the tidal ranges. As the new configuration of the estuary developed under the influence of the altered conditions, the level of high water gradually increased, until after a time corresponding to 40 years of tides, when the estuary had become stabilized, the mean level of high water was 0·4 foot higher than before the introduction of the obstruction, while the level of low water was 0·3 foot higher. Under these conditions the scour at the bridge had increased the area of waterway until the percentage obstruction at H.W. amounted to approximately 66 per cent. of the cross section of the original waterway.

Observation showed that the time of high water at spring tide in the estuary is later with the embankment in existence, there being a delay of 37 minutes at Heswall, 59 minutes at Pentre, and 67 minutes at Connah's Quay.   At neap tides observation showed that the mean high-water level was 0·2 foot higher with than without the embankment, while the low-water level was not appreciably altered.   At the time these latter results were looked upon as being somewhat anomalous, but as it became necessary to remove the model, the matter was not pursued further.

EXPERIMENTS ON A SMALL MODEL OF THE SEVERN ESTUARY.

When, recently, the present investigation was begun, neither the Dee model nor the original large model of the Severn was available, the latter having been dismantled some years previously.   A very

Fig. 2.

Scale: 1 inch = 10 miles.

SCOPE OF SEVERN TIDAL MODEL.
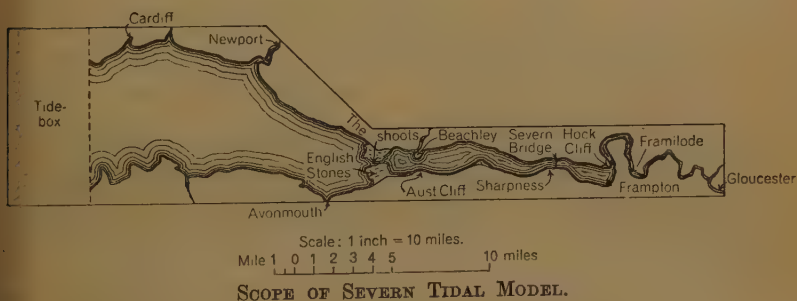
small model of the Severn, which had been constructed with a view to demonstrating the general working of such models rather than for experimental work, was, however, available, and it was decided to explore its possibilities.   The model is shown in *Fig. 2*, and includes the estuary from Gloucester to a point 14 miles seaward of Avonmouth.   The horizontal-scale ratio is 1 : 40,000, the vertical-scale ratio is 1 : 366, and the time-scale ratio is 1 : 2,090.   The overall length of the model is 6 feet.

The method of visual observation of the water-levels against graduated tide-gauges, which had been used to obtain tidal data in the larger model with its larger and slower tides, was useless for the purpose of obtaining sufficiently accurate measurements, but it was found that by using a micrometer depth-gauge with a needle-point extension, and by observing through a magnifying lens, changes in the level of high water amounting to 0·001 inch, corresponding to

0·4 inch in nature, could readily be measured, while changes of one-half of this magnitude could be detected.

In order to check the accuracy of the model in reproducing the tidal phenomena in the open estuary, observations of the levels of high water and of the tidal range at spring tides were made at a series of points.   The range of normal spring tides at Avonmouth is 40·4 feet, corresponding to 1·33 inch in the model.   The mechanism was set to give the correct high-water level and range at Avonmouth, and the values obtained after some adjustments are shown in the following Table, together with the corresponding figures for the estuary given by the Admiralty and appearing in the Appendix to the Report of the Sub-Committee of the Severn Barrage Committee.[1]

		Avon-mouth.	Beachley.	Severn Bridge.	Frami-lode.	Glouces-ter.
Level of high-water spring tides : feet above O.D.	Model	22·5	23·2	25·4	26·9	26·1
	Estuary	22·5	23·4	25·5	27·0	26·0
Range : feet . .	Model	40·4	41·4	28·8	11·3	6·6
	Estuary	40·4	41·0	28·9	11·5	6·5

As a further check on the accuracy of the model, a series of float observations was made, in which floats were released at the seaward end of the Shoots at the beginning of a spring flood tide.   Observations by the Admiralty, recorded in the Report of the Severn Barrage Committee, show that such floats travelled upstream to Framilode, a distance of some 33 miles, the time being 5 hours 52 minutes.   In the model the floats travelled to within ½ inch of the corresponding point and the mean time was 10·0 seconds, corresponding to 5 hours 48 minutes in nature.   Further observations showed that a float released at Framilode at high-water spring tides travels downstream to Severn Bridge in 6·60 seconds, corresponding to 3 hours 50 minutes in nature.   Corresponding Admiralty observations in the estuary gave a time of 3 hours 54 minutes.

The general effect on the tidal phenomena of the natural obstructions and resistances in the estuary is so closely reproduced in the model as to justify the conclusion that the effect of any added artificial obstruction and resistance will be equally well reproduced.

Following this calibration of the model a series of obstructions, of the nature of bridge-piers, was placed across the English Stones, in a line parallel to and about 500 feet seaward of the line of the Severn

tunnel.   This is the site originally chosen for the suggested Severn bridge.   Here the tidal range at springs is approximately 41 feet, high-water level being $+22.5$ O.D.   The average level of the English Stones at the site is -7.5 O.D.   The cross section is approximately as shown in *Fig. 3.*   The Stones are uncovered for about 2 hours on either side of low-water springs, and the last of the ebb discharge takes place through the narrow rock gorge called the Shoots.   The cross-sectional area at high water is some 460,000 square feet, and the area of the estuary at high water, above the line of the proposed bridge, is 34.8 square miles.

The obstruction offered by the piers of the proposed bridge was 8 per cent. of the cross section at high water, but experiments were carried out in the model with obstructions ranging up to 60 per cent. of the cross section.   The smaller obstructions—those up to 20 per

Fig. 3.

H.W. Springs

Sudbrook

(Monmouthshire) bank.

New Passage
(Gloucestershire) side

Ordnance datum

L.W. Springs

English Stones

The Shoots

Horizontal scale: 1 : 44,000

Feet 1,000 500 0   1,000   2,000   3,000   4,000 feet

Vertical scale: 1 : 1,000.

Feet 25   0   25   50   75   100 feet

SCALE OF DEPTHS: FEET.

+ 20

0

- 20

- 40

- 60

- 80

- 100
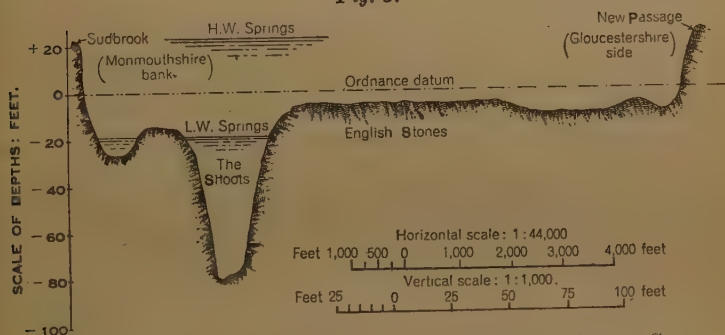
CROSS SECTION OF RIVER SEVERN AT THE ENGLISH STONES SITE.

cent.—consisted of a series of equally-spaced bridge-piers of elliptical section, with the longer axis approximately 3 times the shorter.   The larger obstructions were pier-shaped masses placed on the English Stones in the line of the proposed bridge.

In each case measurements were made of the levels of high and low water at Avonmouth, Beachley, off Frampton, and at Gloucester, and of the times of high water at Avonmouth, Beachley and Gloucester, with and without the obstruction in position.

The main results of the observations are shown in the Table on p. 216 and in *Fig. 4* (p. 216), which also shows the results of the tests on the Severn Barrage model.

In this Table, positive values represent an increase in the level of high water.

The experiments showed the unexpected result that for obstructions of less than 35 per cent. the level of high water everywhere

Obstruction of water-way : percentage of cross section.	Effect on level of high-water spring tides : inches in nature.				Delay in time of high-water slack : minutes in nature.		
	Avon-mouth.	Beachley.	Framp-ton.	Glouces-ter.	Avon-mouth.	Beachley.	Glouces-ter.
7·5	Small.	Small.	+0·4	+0·6	0·4	1·6	1·0
10·0	Small.	Small.	+0·6	+0·9	0·6	2·0	1·2
15·0	−0·3	+0·2	+1·5	+2·5	1·1	3·3	2·0
20·0	−0·5	+0·3	+3·3	+5·0	2·0	5·5	2·9
25·0	−0·6	+0·6	+6·0	+7·8	3·0	8·3	4·2
30·0	−0·7	+0·5	+8·5	+10·6	4·0	12·0	6·2
35·0	−0·6	0·0	+10·1	+12·2	6·0	17·3	8·4
40·0	−0·5	−1·0	+10·5	+12·4	8·0	23·4	11·2
50·0	0·0	−3·5	+7·0	+ 9·5	14·5	40·0	18·3
60·0	−1·0	−7·5	−7·5	−7·0	20·6	59·0	28·1

Fig. 4.
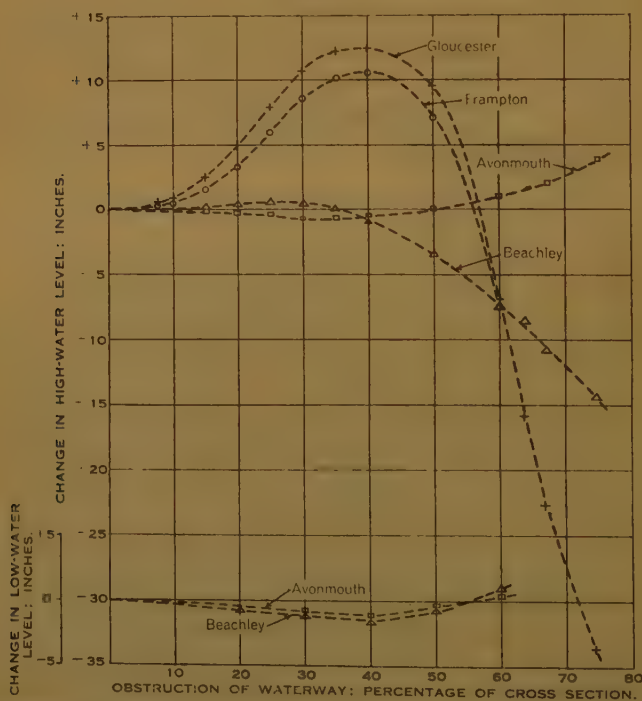
EFFECT OF OBSTRUCTIONS AT THE ENGLISH STONES ON THE LEVELS OF HIGH- AND LOW-WATER SPRING TIDES.

upstream is higher with than without the obstruction. The effect is greatest at the upper end of the estuary, where it is a maximum

(12·4 inches) with a 40-per-cent. obstruction.   The downstream high-water level at Avonmouth is only very slightly changed with obstructions less than 50 per cent., and is increased with greater obstructions. The effect on the level of low water is very small.   At Avonmouth it is lowered slightly with obstructions of up to 50 per cent., while at Beachley, above the obstruction, it is lowered slightly with small obstructions, and higher up the estuary at Frampton and Gloucester the effect is negligible.

In view of the nature of the results it was decided to repeat the experiments with piers at another section of the estuary, and these were installed along a line between Beachley and Aust Cliff.   At this site, which is about 2½ miles above the English Stones, the width of the channel is reduced to 1 mile.   The area of the waterway at high water is about 230,000 square feet.   The area of the upstream estuary is 28·3 square miles at high water and the volume passing at spring tides is some 18,500 million cubic feet, or 75 per cent. of that passing the English Stones, so that mean velocities are some 50 per cent. greater than at the latter site.   The Beachley site differs from the English Stones in that as the piers are founded in the bed of the channel their effect is felt down to low water.   Also, whereas at the Stones the piers rest on a rock plateau, so that the question of erosion of the bottom or movement of sandbanks in the vicinity does not arise, there are sandbanks at the Beachley site.   To prevent complications due to any erosion of the bed altering the cross-sectional area, it was stabilized for the purpose of these experiments by a light wash of weak cement.

The results of these tests are given in the Table on p. 218 and in *Fig. 5*, p. 219.

The general results are of the same kind as in the previous experiments.   The maximum effect, at Gloucester and Frampton, is somewhat greater and is attained with a greater obstruction—about 50 per cent.   With this the high-water level is raised 14·6 inches at Gloucester, 12·0 inches at Frampton, and 1·5 inch at Beachley. Low-water level at Beachley is raised 3·0 inches, at Avonmouth it is lowered 0·6 inch, while at Frampton and Gloucester it is not appreciably altered.

It was next decided to attempt to obtain tide-curves for the estuary, both without any obstruction and with this 50-per-cent. obstruction. No difficulty was experienced in obtaining a series of points on the flood-tide curve.   The needle of the micrometer having been adjusted to some predetermined level between high and low water, the instant at which the rising tide reached this was recorded by chronometer, the datum time being fixed by the instant at which a point on the driving disc of the tidal mechanism passed a given mark.   The

Obstruction of waterway: percentage of cross section.		7·5	10	15	20	30	40	50	60	70
Effect on level of high-water springs: inches.	Avonmouth	†−v.s.	−v.s.	−v.s.	−0·2	−0·6	−1·5	−1·2	−0·4	+1·0
	*Beachley	+v.s.	+v.s.	+0·2	+0·4	+1·0	+1·8	+1·5	−0·4	−3·5
	Frampton	+v.s.	+0·2	+1·0	+3·2	+8·0	+11·3	+12·0	+8·6	−4·0
	Gloucester	+v.s.	+0·3	+1·6	+4·4	+10·4	+14·1	+14·6	+11·5	−3·0
Effect on level of low-water springs: inches.	Avonmouth	−v.s.	−v.s.	−v.s.	−0·4	−0·6	−1·0	−0·6	0·0	+1·3
	*Beachley	v.s.	v.s.	+0·3	+0·5	+1·0	+2·0	+3·1	+4·5	+6·2
	Frampton					very small			+0·4	+0·5
	Gloucester					very small				
Delay in time of high-water slack: minutes.	Avonmouth	v.s.	0·4	1·0	2·0	4·0	6·5	9·5	14·4	20·0
	*Beachley	0·9	1·2	3·0	4·8	11·2	21·0	35·0	51·5	70·0
	Frampton	v.s.	0·9	2·0	3·9	8·6	15·1	25·0	36·0	48·5
	Gloucester	v.s.	0·6	1·2	2·4	5·6	10·8	17·5	26·0	35·5
Delay in time of low-water slack: minutes.	Avonmouth	v.s.	0·4	1·1	1·5	2·6	4·1	6·0	8·4	10·8
	*Beachley	0·9	1·2	2·0	3·0	5·5	8·6	12·5	18·0	21·6
	Frampton	v.s.	0·9	1·3	2·2	3·8	6·0	9·4	13·0	16·6
	Gloucester	v.s.	0·6	1·2	1·9	3·1	5·0	8·0	11·1	14·8

* The Beachley figures refer to a point 400 yards above the obstruction.   † " v.s." signifies " very small."

mean of a series of four observations at each height was found to give a value which could be repeated without difficulty. More difficulty was experienced on the ebb tide owing to the formation of a meniscus which clung to the gauge-point. The method finally adopted was to observe the times at which the meniscus broke away from the point. Observation showed that when the point was raised from below the surface of still water the meniscus broke away when

Fig. 5.
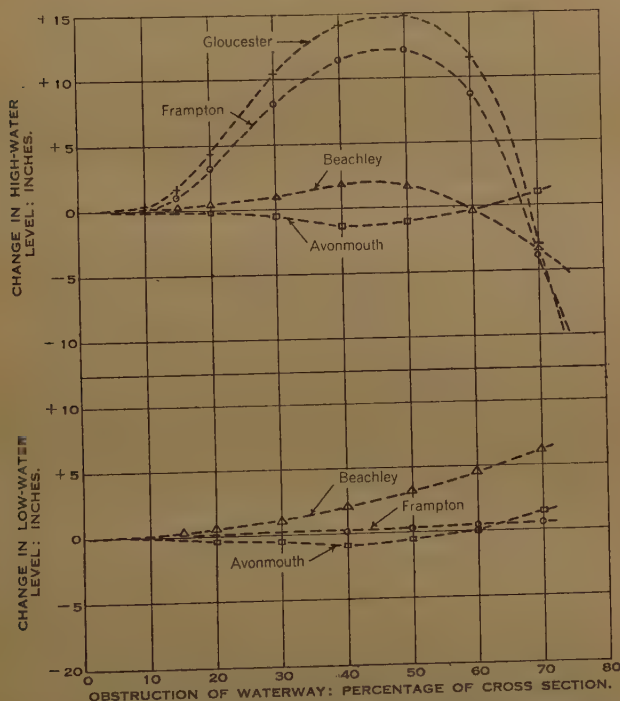
EFFECT OF AN OBSTRUCTION AT BEACHLEY ON THE LEVELS OF
HIGH- AND LOW-WATER SPRING TIDES.

the point was 0·025 inch above the level of the surface, and in plotting the tide-curves each observed level on the ebb has been lowered by this amount. This correction renders the ebb-curves less reliable than the flood-curves. Any error involved will, however, be sensibly the same for corresponding observations with and without the obstruction in position, and the relative values should not be affected.

In the experiments tide-curves were obtained at Avonmouth, Beachley (about ¼ mile above and below the obstruction), Sharpness, and Gloucester. These are shown in *Figs. 6* (pp. 220–1), on a scale

Figs. 6.

(a)
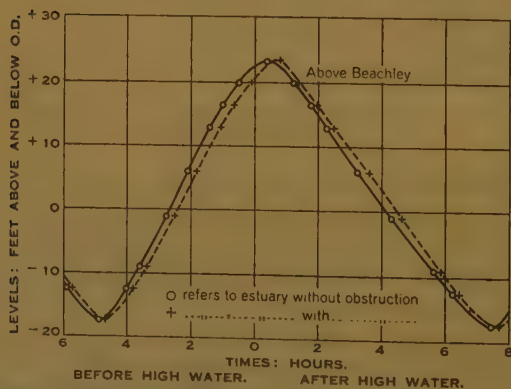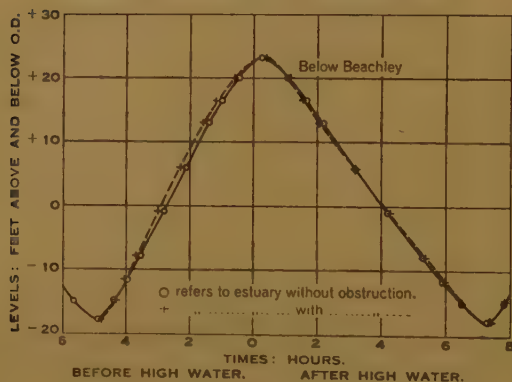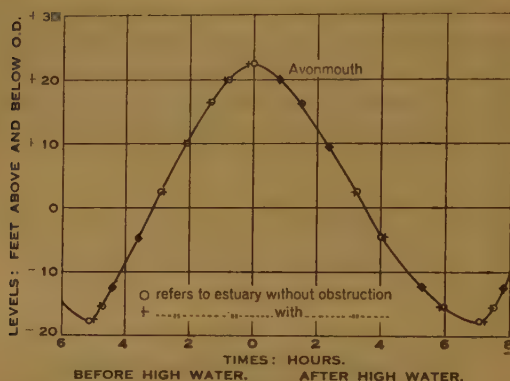
TIDE-CURVES WITH AND WITHOUT A 50-PER-CENT. OBSTRUCTION
AT BEACHLEY.

Figs. 6.

(b)

TIDE-CURVES WITH AND WITHOUT A 50-PER-CENT. OBSTRUCTION
AT BEACHLEY.

15
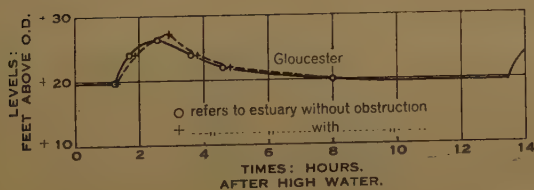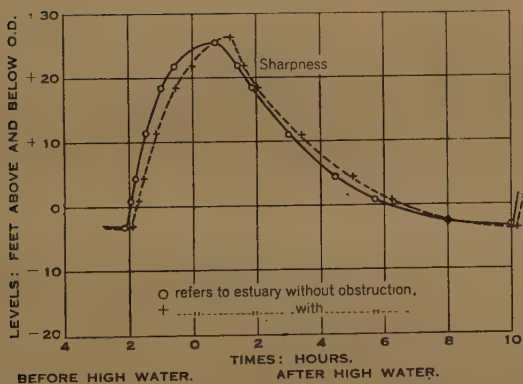
representing corresponding tidal levels and times in the actual estuary. At Avonmouth the two curves are almost identical. Below Beachley the effect of the obstruction is mainly felt on the flood tide, which rises relatively more quickly when the estuary is obstructed. Above Beachley and at Sharpness the effect is much greater, and there is a definite lag which becomes most pronounced during the latter half of the flood tide.

At first sight the fact that the introduction of an obstruction, with the additional resistance which it causes, is accompanied by an increase in the tidal range and in the volume of water entering the upper estuary would appear to involve a violation of laws of the conservation of energy. An analysis of the tide-curves, however, shows that this is not the case. If the surface-levels at any tide-gauge station are measured at equal intervals of time, and if the heights above low-water level are measured at the middle of each interval, the sum of the products of the height and the rise in level during any interval is a measure of the potential energy per unit of surface-area at that station. The sum of such products from low water to high water is a measure of the energy of the flood tide, and that from high water to low water is a measure of the energy of the ebb tide.

If the tide-curves obtained with and without the 50-per-cent. obstruction at Beachley are analysed in this way, the following results are obtained :—

Site.	Potential energy per square foot of surface-area : ft.-lb.			
	Flood tide.		Ebb tide.	
	Normal.	With 50-per-cent. obstruction.	Normal.	With 50-per-cent. obstruction.
Avonmouth	55,600	55,600	53,300	53,300
Below Beachley . . .	55,400	55,000	54,000	53,500
Above Beachley . . .	54,500	52,000	53,200	51,700
Sharpness	30,300	29,500	27,400	26,900

The figures give the energy in foot-lb. per square foot of surface-area in the estuary at the section in question. Taking the flood-tide observations as being the more reliable, they show that the tidal energy of a flood tide in the upper estuary is reduced by about 4 per cent. by the introduction of the obstruction, the effect being less at Sharpness than lower down the estuary at Beachley.

TESTS ON A MODEL OF A SYMMETRICAL TIDAL ESTUARY.

It was next decided to carry out similar tests on another form of estuary in order to ascertain whether or not the phenomenon just described is due to the peculiar conformation of the Severn estuary, and a model was constructed of a symmetrical tidal estuary of the dimensions and proportions shown in *Fig. 7.* This was made of

Fig. 7.
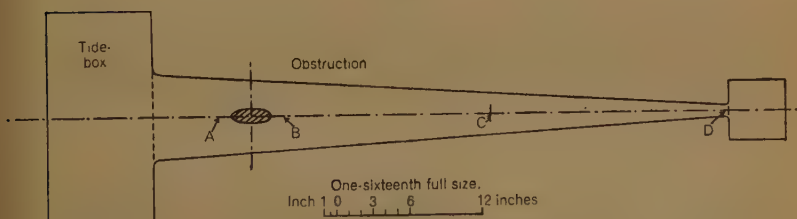
PLAN OF SYMMETRICAL-ESTUARY MODEL.

galvanized iron with a smooth and level bed. A series of obstructions was placed at a point 40 inches from the top of the estuary, where the width is 6 inches. The effect on the level of high water at the top of the estuary is as follows :—

Obstruction of waterway : percentage of cross section.	20	30	40	50	60	70	80
Change in level at top of estuary (actual): inch	+0·0025	+0·0050	+0·0105	+0·0210	+0·0340	+0·0260	+0·0040

Assuming this to represent an estuary to the same scales as the Severn model, the maximum effect on the high-water level is equivalent to an increase of 12·4 inches. This maximum effect is obtained with a 60-per-cent. obstruction. Tide-curves were taken with and without this obstruction in place at the following points (*Fig. 7*) : below the obstruction (A) ; at the upper end of the estuary (D) ; and half-way between the obstruction and the upper end (C). These tide-curves are shown in *Figs. 8*, p. 224.

Figs. 8.
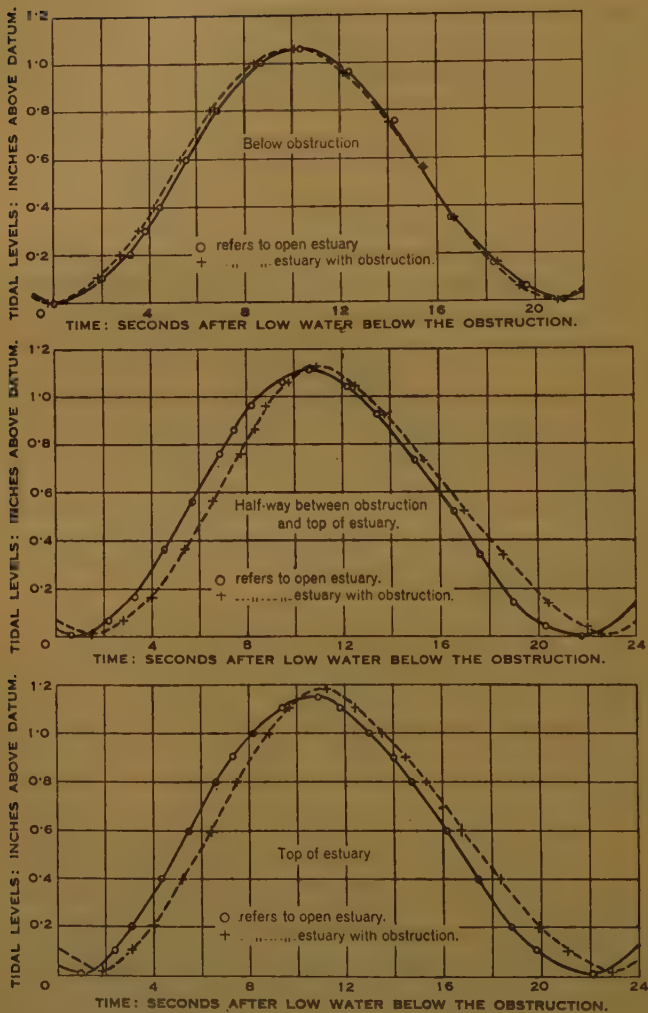
TIDE-CURVES FROM SYMMETRICAL-ESTUARY MODEL WITH AND
WITHOUT A 60-PER-CENT. OBSTRUCTION.

The observations show the following results :—

Site.	Range of tide : inch.		Relative levels with 60-per-cent. obstruction.	
	Without obstruction.	With obstruction.	High water.	Low water.
Seaward end of estuary .	0·976	0·976	0·00	0·00
Just below obstruction .	1·060	1·060	—0·003	—0·003
Just above obstruction .	1·070	1·069	0·0	+0·001
Half-way between obstruction and top of estuary	1·108	1·115	+0·012	+0·005
Top of estuary . . .	1·180	1·204	+0·034	+0·010

This model differs essentially from that of the Severn in having a level bed, with the result that the range of tide is greatest at the upper end of the estuary. The results of an obstruction are, however, of the same general kind. An analysis of the tide-curves gives the following results, the units being arbitrary :

Site.	Tidal energy per unit area.			
	Flood.		Ebb.	
	Without obstruction.	With obstruction.	Without obstruction.	With obstruction.
Below obstruction .	57·4	57·4	56·2	55·7
Just above obstruction . . .	54·0	51·8	52·9	51·4
Half-way along tidal basin	65·1	64·0	62·5	62·0
At upper end . .	66·0	65·2	63·9	63·4

In this case the mean energy of a flood tide is approximately 2·5 per cent. less with the obstruction than without it.

In these experiments the depth of water at high water was 2·35 inches at the seaward end and 2·47 inches at the top of the estuary. With the same depth of water, but with the bottom covered with a layer of sand of medium coarseness, the effect of the obstruction was greater, the level of high water at the top of the estuary being raised by the equivalent of 13·7 inches as compared with the equivalent rise of 12·4 inches with the smooth bottom.

With a sloping sandy bottom, and with the same depth of water at the seaward end and a depth of 1·5 inch at the top of the estuary, the same obstruction raised the level of high water at the top of the

estuary by the equivalent of 14·6 inches.   When the general level of the water was lowered 0·25 inch, the obstruction raised the level of high water by the equivalent of 16·4 inches.

FURTHER EXPERIMENTS ON THE SEVERN MODEL.

A further series of tests was next carried out on the Severn model with various modifications.

In the first of these, a 20-per-cent. obstruction was placed on the English Stones, together with a 30-per-cent. obstruction at the Beachley site.   The effect of this combination was to increase the high-water level at Beachley by 1·3 inch, that at Frampton by 10·6 inches, and that at Gloucester by 14·5 inches.   With the 30-per-cent. obstruction at Beachley alone, these figures were respectively 1·0 inch, 8·0 inches and 10·5 inches, while with the 20-per-cent. obstruction at the English Stones they were 0·5 inch, 3·4 inches and 5·0 inches, so that the effect of this particular combination was approximately equal to the sum of the effects of its components.

In a second experiment the estuary was blocked at Hock Cliff, cutting off the 20 miles of river channel between this point and Gloucester.   With a 50-per-cent. obstruction at Beachley the high-water level just above Beachley was 4·4 inches higher than without the obstruction, while at Frampton the high-water level was raised 4·8 inches.   These figures compare with 1·5 inch and 12·0 inches when the estuary is open to Gloucester.

The effect of removing the Beachley peninsula was tried in a third experiment.   This increases the area of the waterway at Beachley by 60 per cent.   As a result the level of high water was lowered at all points in the upper estuary, by 1·2 inch at Beachley, by 5·5 inches at Frampton, and by 6·6 inches at Gloucester, while the level of low water at Beachley was very slightly lowered.   In this estuary (that is, without the Beachley peninsula), the maximum increase in the level of high water upstream, due to an obstruction at Beachley, is produced by an obstruction equivalent to the peninsula plus 50 per cent. of the original waterway, which together are equivalent to 70 per cent. of the enlarged waterway.   This obstruction raises the high-water level at Gloucester by 21·2 inches, that at Frampton by 17·5 inches, and that at Beachley by 2·8 inches.

In a fourth experiment, the estuary was restored to its normal condition, and the effect of reducing the area of the waterway at Sharpness was examined.   Here all reductions of area amounting to less than 25 per cent. slightly increased the level of high water at Gloucester.   With a 25-per-cent. obstruction the level was the same

as without the obstruction. With a 50-per-cent. obstruction the high-water level at Gloucester was lowered by 1·5 inch.

In a fifth experiment, the speed of the tidal gearing was reduced so as to give a tidal period of 40·80 seconds instead of 21·15 seconds. This naturally changed the tidal characteristics and reduced the level of high water at all points above Avonmouth. Under these conditions a 50-per-cent. obstruction at Beachley raised the high-water level at Frampton by 2·9 inches and that at Gloucester by 3·8 inches. These figures compare with 12·0 inches and 14·6 inches with the normal tidal period. The squares of the tidal periods are in the ratio 3·7 to 1, so that the ratio of the effects is approximately the same as that of the inverse square of the periods and therefore of the velocities.

A similar experiment with a 40-per-cent. obstruction at the English Stones showed that this raised the high-water level at Frampton by 2·4 inches, as compared with a rise of 10·5 inches with the normal tidal period. Here again the ratio is approximately the same.

DISCUSSION OF RESULTS.

The results observed in these various experiments would appear to be due to partial resonance.

The mass of water in the upper estuary forms a system which, if suddenly isolated, would surge to and fro until it attained its equilibrium level. The period of this surge is less than the tidal period. This is acted upon by a periodic force—the tidal impulse—whose magnitude varies with time, approximately according to a cosine law. The motion is opposed by various hydraulic resistances, and the resultant tidal phenomena in the upper estuary are largely governed by the interaction of these forces.

The question is too complex for mathematical analysis, but some light on the problem may be obtained by considering the analogy of a solid system capable of natural vibration, acted upon by a periodic force obeying a cosine law, and having its motion resisted by a frictional force. If this latter force is proportional to the velocity the problem is capable of mathematical solution.

The equation of motion becomes of the form

$$\frac{w}{g}\frac{d^2x}{dt^2} + \mu\frac{dx}{dt} + Sx = Q\cos\alpha t,$$

where w denotes the weight of the vibrating body,
  g „   acceleration due to gravity,
  x „   displacement of the vibrating body,
  t „   time,
  μ „   frictional coefficient,

S denotes the elastic restoring force when the displacement x
is unity,

$2\pi/\alpha$,,   period of the applied force,

and Q ,,   maximum value of the applied force :

or
$$\frac{d^2x}{dt^2} + p\frac{dx}{dt} + f^2x = q \cos \alpha t,$$

where $2\pi/f$ denotes the period of the natural oscillation ;
and q is a constant.

The ultimate displacement x from the equilibrium position a
time t is then given by

$$x = \frac{q}{\sqrt{(f^2 - \alpha^2)^2 + \alpha^2 p^2}} \cdot \cos(\alpha t + \gamma),$$

where γ denotes $\tan^{-1}\dfrac{\alpha p}{f^2 - \alpha^2}$, and is the difference of phase between
the vibration of the body and that of the applied force.

The semi-amplitude of the resultant vibration is

$$\frac{q}{\sqrt{(f^2 - \alpha^2)^2 + \alpha^2 p^2}}.$$

With a given applied force, and with a system in which the natural
period of vibration is independent of the frictional resistance, any
increase in this resistance would reduce the amplitude, but if the
increase in frictional resistance increased the time of natural vibration
(that is to say, if it reduced f), the amplitude might be either increased
or reduced, depending on the relative values of f, α, and p.

The state of affairs in the actual problem naturally differs in
important respects from that of this elastic vibrating system.   The
natural vibration of the estuarine column does not consist of a simple
single surge from end to end, but is very complex, and its period
varies somewhat at different points in the estuary ; nor is the lag, or
difference of phase between the tidal oscillation in the upper estuary
and that in the lower estuary, the same at all points, or indeed the
same at high water as at low water.

Also, in the estuary and model the resistance is proportional to v^n,
where n is approximately equal to 2, instead of being proportional
to v.   In such a case, however, experience on other problems involving
surges (such as occur, for example, in the pipe-lines of hydraulic
power-plants fitted with surge-tanks) shows that a close approxima-
tion to the actual results may be obtained by assuming that the
resistance is proportional to v, and by adopting a value for the

frictional coefficient which will make the calculated value of the mean friction over the given range of velocities equal to its true mean value.

Observation shows that the introduction of an obstruction in the upper estuary increases the period of the natural oscillation in this part of the estuary, although that in the open estuary below Avonmouth is almost unaltered. The following Table shows the period, in seconds, of this surge both in the normal estuary and with a 50-per-cent. obstruction at Beachley :—

Site.	Period of surge : seconds.	
	Normal estuary.	With 50-per-cent. obstruction.
At seaward end of model . . .	6·65	6·65
Avonmouth	6·58	6·60
English Stones	6·32	6·58
Beachley	6·20	6·48
Sharpness	6·12	6·40

The obstruction also causes a lag in the times of high and low water in the upper estuary, and thereby increases the difference of phase between the tidal oscillations above and below the obstruction.

Now, although the analogy with the simple vibrating system should not be carried too far, the simple analysis does enable some sort of calculation to be made of what might be expected to be the relative effect of obstructions of different amounts on the tidal amplitudes, in terms of the observed mean lags and periods, provided that the effect is due to resonance. Thus the curves of *Fig. 5* indicate that practically the same mean amplitude (tidal range) is obtained at points above Beachley with a 66-per-cent. obstruction at Beachley as when the estuary is open. Observation also shows that this obstruction causes a mean lag of 0·88 second in the time of the turn of the tide in the upper estuary, and since the time of a complete tide is 21·15 seconds the equivalent phase-lag is $\dfrac{0.88 \times 360}{21.15}$ degrees = 15 degrees greater when the obstacle is in position.

Observation also shows that the mean period of natural oscillation of the column in the upper estuary is 6·55 seconds when the obstruction is in position, and 6·16 seconds when it is removed. If the suffix o refers to the estuary without the obstruction,

then $\qquad \dfrac{2\pi}{f_o} = 6.16, \ \dfrac{2\pi}{f} = 6.55, \text{ and } \dfrac{2\pi}{\alpha} = 21.15,$

so that $\qquad f_0 = 1\cdot02, f = 0\cdot96$, and $\alpha = 0\cdot297$.

Since the amplitude is the same in each case, and since the value of the applied force is the same in each,

$$(f^2 - \alpha^2)^2 + \alpha^2 p^2 = (f_0^2 - \alpha^2)^2 + \alpha^2 p_0^2.$$

Substituting the above values for f_0, f, and α gives

$$p^2 - p_0^2 = 2\cdot42.$$

But $p = \dfrac{f^2 - \alpha^2}{\alpha} \cdot \tan\gamma$, and $p_0 = \dfrac{f_0^2 - \alpha^2}{\alpha} \cdot \tan\gamma_0$,

whence $\qquad p^2 - p_0^2 = \left(\dfrac{f^2 - \alpha^2}{\alpha}\right)^2 \tan^2\gamma - \left(\dfrac{f_0^2 - \alpha^2}{\alpha}\right)^2 \tan^2\gamma_0$,

which, on substituting the values of f_0, f, and α, gives

$$p^2 - p_0^2 = 7\cdot89 \tan^2\gamma - 10\cdot30 \tan^2\gamma_0.$$

But with a 66-per-cent. obstruction $\gamma - \gamma_0$ is 15 degrees and

$$p^2 - p_0^2 = 2\cdot42,$$

whence $2\cdot42 = 7\cdot89 \tan^2(\gamma_0 + 15 \text{ degrees}) - 10\cdot30 \tan^2\gamma_0$.

The solution of this equation by trial and error gives $\gamma_0 = 19$ degrees 30 minutes, so that $\gamma = 34$ degrees 30 minutes.

From this $\qquad p = \dfrac{f^2 - \alpha^2}{\alpha} \tan\gamma = 1\cdot93$,

and $\qquad p_0 = \dfrac{f_0^2 - \alpha^2}{\alpha} \tan\gamma_0 = 1\cdot145$;

p and p_0 are measures of the resistance to flow with and without the obstruction in position. They indicate that this obstruction increases the overall resistance by 68 per cent.

Considering now the case of a 50-per-cent. obstruction, if it is assumed that the resistance due to this is less than that of the 66-per-cent. obstruction in the ratio of $\left(\dfrac{50}{66}\right)^2$, this makes the new value of

$$p - p_0 = 0\cdot785 \times \left(\dfrac{50}{66}\right)^2 = 0\cdot451 ;$$

but $\qquad p_0 = 1\cdot145$

whence $\qquad p = 1\cdot596$.

Observation shows that $\gamma - \gamma_0$ now equals 9 degrees 15 minutes and therefore $\gamma = 28$ degrees 45 minutes, while the natural period is 6·44 seconds so that $f = \dfrac{2\pi}{6\cdot44} = 0\cdot976$, and f_0 as before is 1·02.

Then $\dfrac{\text{amplitude with obstruction}}{\text{amplitude without obstruction}} = \sqrt{\dfrac{(f_0{}^2 - \alpha^2)^2 + \alpha^2 p_0{}^2}{(f^2 - \alpha^2)^2 + \alpha^2 p^2}}$

$$= 1\cdot027.$$

A similar investigation for a 30-per-cent. obstruction, where $\gamma - \gamma_0 = 3$ degrees 30 minutes, the natural period is 6·28 seconds, and $p = 1\cdot306$, makes the ratio of the amplitudes with and without the obstruction equal to 1·020.

With a 75-per-cent. obstruction $\gamma - \gamma_0$ is 18 degrees 30 minutes, the natural period is 6·60 seconds, and the value of p is 2·165 ; making the calculated mean amplitude 3·2 per cent. less than without the obstruction.

The relations between the observed effects of these obstructions on the amplitude and those calculated in this way from the observed lags and periodicities are shown in the following Table :—

Obstruction : percentage of waterway.	Effect on mean tidal range above obstruction : per cent.	
	Observed.	Calculated.
30	+1·8	+2·0
50	+2·4	+2·7
75	−3·5	−3·2

CONCLUSIONS.

The results indicate that an increased constriction in the waterway of a tidal estuary does not necessarily involve a diminution in the volume entering and leaving the upper estuary per tide ; on the contrary, in estuaries of the type and proportions examined, a constriction of section up to a certain point increases the level of high water at all points above the constriction and increases the volume entering and leaving.

The effect of considerable percentage constrictions on the levels of high water and low water and on the tidal range is shown to be somewhat surprisingly small. Thus, in the case of the Severn estuary with an artificial obstruction at Beachley, the effect on the mean tidal range in the upper estuary at spring tides was less than 3 per cent. so long as the obstruction did not exceed 70 per cent. of the waterway.

With such percentage obstruction as would be caused by the piers of any normal bridge, the effect on the tidal levels and on the volume of flow is so extremely small that it is inconceivable that it could

have any but a local effect on the tidal scour.   Where, as in the cases examined, such an obstruction increases the volume of flow, its effect, if any, should be beneficial rather than the reverse.

The results would also appear to be of interest as indicating that, for such an investigation as that in question, where movement of bed-materials is not involved, a comparatively small-scale model is capable of giving results with a sufficiently high degree of accuracy. They also point to the danger of attempting to predict, without the use of a model, the probable effect of any modification in the case of such a complex system of forces and interactions as occurs in estuarine flow.

ADDENDUM.

Since the previous section of the Paper was written, the following additional evidence bearing on the question has become available.

Experiments on a Small-Scale Model of the Estuary of the Dee.

A model to a horizontal scale of 1 : 40,000 and a vertical scale of 1 : 400 has been constructed of the estuary of the Dee.   This includes a portion of Liverpool Bay extending 4·5 miles seaward of a line joining Point of Air and Hilbre.   It differs from the larger model of the Dee, previously referred to, in that the latter also included the estuary of the Mersey and a much larger area of Liverpool Bay.

Experiments on this smaller model show that an obstruction similar to that in the larger model, and obstructing 74 per cent. of the waterway, lowered the mean high-water level at spring tides by 1·0 foot, whilst a 66-per-cent. obstruction raised the high-water level by 4 inches, results which are in agreement with those from the larger model.   An obstruction of 35 per cent. blocking up the Hilbre Swash on the northern side of the entrance increased the high-water level at Connah's Quay by 8·0 inches, whilst an obstruction of the same size blocking the entrance to the Mostyn Deep on the southern side of the entrance had no measurable effect on the level.   The two obstructions together lowered high-water level by 4·5 inches.

Experiments on a Large-Scale Model of the Estuary of the Dee.

Experiments are at the moment in progress in the Author's laboratory on a larger-scale model of the Dee to a horizontal scale of 1 : 5,000 and a vertical scale of 1 : 200.   The effects of certain schemes of double training walls below Connah's Quay have been investigated. In one such scheme the reduction of the width between the walls

from 600 feet to 300 feet increased the high-water level at Connah's Quay by 0·7 foot without affecting the level of low water.

Experiments on a Model of the Estuary of the Humber.

Some 7 years ago a model of the estuary of the Humber, extending from a point 2 miles seaward of Immingham to points on the Ouse and Trent about 20 miles above the junction of these rivers, was constructed in the Author's laboratory for the purpose of investigating the probable effect on tidal levels, currents, and on the conformation of the bed, of the piers of a proposed bridge between Barton and Hessle, about 6 miles above Hull. This bridge was to have sixteen piers, which would cause an obstruction of approximately 4·5 per cent. to the waterway at high water of spring tides. The model had a horizontal-scale ratio of 1 : 7,040 and a vertical-scale ratio of 1 : 192.

Experiments showed that the effect of the piers on the tidal levels was too small to be measured with any accuracy, and the same applied when the piers were widened so as to obstruct 8 per cent. of the waterway.

At a later stage the effect of large obstructions at the bridge-site was investigated. In the first experiment the main span of the bridge, 924 feet in length, was entirely blocked ; following this an additional length of 760 feet on the south side of the main span was also blocked. These obstructions reduced the area of waterway at high water of spring tides by about 30 per cent. and 43 per cent. respectively. The effect of the 30-per-cent. obstruction was to reduce the high-water level at spring tides at Goole by approximately 0·1 foot, and to raise low-water level by the same amount, the effect at points nearer the bridge becoming progressively less. With the 43-per-cent. obstruction the high-water level at Goole was reduced by 0·3 foot, and the low-water level was raised by 0·2 foot.

In this general connexion the following quotation from p. 181 of " Tidal Rivers," by W. H. Wheeler, is of interest :

" At Shoreham, the widening and deepening of the harbour mouth resulted in depressing the rise of spring tides from 18 feet to 16 feet. At Newhaven also the works carried out for improving the entrance to the harbour, although causing the tide to be earlier, have made the rise 2 feet less than formerly."

The Paper is accompanied by eight sheets of drawings, from which the Figures in the text have been prepared, and by one photograph.

Discussion.

The Author.

The AUTHOR exhibited a number of lantern-slides illustrating his Paper, and demonstrated the operation of a tidal model.

Mr. Wilson.

Mr. M. F.-G. WILSON, Vice-President, remarked that his firm in conjunction with Sir John Griffith, Past-President Inst. C.E., had had to examine the Lower Severn in considerable detail some time ago, in connexion with a proposal to construct a barrage for the development of water-power. The barrage was to have been built either at the English Stones or at Beachley; and Professor Gibson had been asked to study the effect of the barrage by means of model-tests. Preference was given to the proposal to place the barrage at the English Stones, which consisted of large shelves of rock exposed at low water across almost the whole width of the river, and which it was intended to utilize for the foundations of the barrage. Passing through those rocks was a deep water channel, known as the Shoots, along whose eastern edge it was intended to construct a masonry dam containing seventy-two turbines having an aggregate output of over 1 million horse-power. Between the turbine-dam and the eastern or left shore there would have been a series of sluices for the regulation of the river. The tide was to have been practically stopped at the barrage, only a comparatively small proportion of the flow passing up and down through the sluices. Professor Gibson's model of the river had worked with extraordinary accuracy, even the Severn Bore being exactly reproduced. Such models represented very accurately the flow of the water, and would give the direction and velocity of currents and the rise and fall of the tide with remarkable precision; it should be realized, however, that they did not give accurate information regarding the actual quantities of shoaling or scouring, though they might give a general idea of where such action would take place and whether it would be of an extensive or slight nature.

Mr. Latham.

Mr. ERNEST LATHAM remarked that he had been interested to note that when Shoreham harbour-mouth had been widened and deepened the high water at spring tides had failed by 2 feet to reach its old level. Whereas widening could obviously cause that effect, it had so far been his experience that deepening of any harbour-mouth or estuary tended to amplify the tidal range and to raise the level of high tide. He would very much appreciate any comments which the Author might care to make on that matter.

Hydraulic models might be misleading. He was dealing with a very interesting legal case in the Fenlands, and, unknown to him, his clients had prepared a model to show that in a steadily-moving stream running at about 1 mile an hour a penstock, which represented about a 30-per-cent. obstruction, would cause a difference of water

level of about 5 feet ! That ridiculous result had arisen because the
flow passed through the model had been far too great in proportion.
The difference of level in actual fact could not have exceeded 3 or 4
inches.   Had that model been produced in court, he did not know
whether it would have convinced the court, or whether the experts
on the other side would have soon demonstrated that the flow was
entirely disproportionate.

Mr. OSCAR ELSDEN asked whether the tide-curves generated by
such small models as that demonstrated by the Author were of the
same extraordinary standard of accuracy as those which the Author
had obtained on the larger-scale models referred to in the Paper.
Had the Author considered studying the results obtained by means
of harmonic analysis ?

It was not stated in the Paper whether the effect of fresh water
flowing down the river had been studied, and he presumed that that
factor had been suppressed. The effect of obstructions on flooding
was, however, one of the bigger implications of the most unexpected
results which the Author had produced, particularly in cases where
a few inches might make all the difference. From some of the curves
given it appeared that an obstruction might hold up the high water
to its original value for about an hour, which might have con-
siderable effects.

He noticed that the model results for the estuaries of the Dee and
Severn, where the tidal ranges were fairly large, showed a rise of
high water produced by an obstruction of the order of 1 foot, whereas
in the case of the South Coast ports of Shoreham and Newhaven,
where the tidal ranges were rather smaller, the removal of obstruc-
tions brought about a fall of 2 feet in the high-water level. The
model seemed to have minimized the effects caused by the obstruc-
tions, and it would be interesting to know whether the Author
considered that to be the case.

Mr. N. G. GEDYE remarked that, without wishing to depreciate
the value or usefulness of experiments on estuary-models, it was not
too much to say that in recent years there had been exhibited in
some quarters a tendency to place too much reliance on them.
The unreliability of the results in certain respects was apparent
from the experience obtained in connexion with several large models
of the Mersey estuary, where the opportunities of comparing indica-
tions obtained from the model with the actual results in nature
had perhaps been more frequent than with any other series of models.
In favourable cases, qualitative indications of the results of pro-
posed works in the estuary could undoubtedly be obtained by
experiment with the model, but any quantitative data so obtained
had to be used with great caution. For instance, the currents
generated in the Mersey models at Liverpool reproduced almost

Mr. Gedye.   identically those actually found, and the models correctly reproduced the general effect of training-walls in the estuary ; they failed however, to give reliable quantitative indications of the effect of scour in channels or the height of the banks formed. It would be unwise to lose sight of that point.

The necessary distortion of scale in estuary-models was only one of several factors which might impair the value of the results. The difference between the horizontal and vertical scales was bound to be great, and the distortion of the scale between the sand used in the model and the actual size of the sand or silt in nature was perhaps of equal consequence. The late Sir Frederick Palmer, Past-President Inst. C.E., in a discussion 7 years ago of a Paper on the Bombay model,[1] had pointed out that the particles used in the model to represent the silt of the harbour were, according to the scale, roughly equivalent to boulders 3 feet in diameter. Sir Frederick had based his objection to scale models mainly on that argument, but Mr. Gedye thought that very few people who were qualified by experience and knowledge to form an opinion would now take the view which Sir Frederick Palmer had then expressed as to the reliance to be placed on model experiments on estuaries.

He believed that the Author's original model of the Severn estuary showed that the structure of the barrage proposed to be made at English Stones would result in the height of high water at spring tides being increased at Avonmouth by about 5 inches and very slightly reduced at Beachley.[2] The figures given by the Author in the Table on p. 218 of the present Paper showed that according to the experiments made on the later model a 70-per-cent. obstruction of the waterway at English Stones would result in an increase in the height of spring tides at Avonmouth of about 1 inch, and a depression at Beachley of about 3·5 inches, whereas with a 50-per cent. obstruction the conditions appeared to be reversed, the water level at Avonmouth being 1·2 inch lower and at Beachley 1· inch higher than the levels under unobstructed conditions. The Severn barrage was supposed to be a solid structure pierced by numerous sluiceways through which tidal water could be allowed to flow. Had the model been operated under the conditions of 100-per-cent. obstruction, as with the sluices closed ? In such conditions, would the Author expect the high-water level below the barrage to be raised or lowered ?

The unexpected results from the Severn model experiments given on pp. 215 and 216 were of very great interest, but the explanation suggested by the Author of the observed phenomenon did not seem

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[1] Minutes of Proceedings Inst. C.E., vol. 232 (1930–31, Part 2), p. 82.
[2] A. H. Gibson, " Construction and operation of a tidal model of the Severn Estuary," p. 225. H.M. Stationery Office, London, 1933.

o be entirely conclusive, and it would be interesting to know whether Mr. Gedye.
e had considered the effect on the experimental results of the dis-
repancy between the time-scale of the model and the time of the
idal flow in nature. He wondered whether the apparent eccentricity
f the tidal levels of the model might not in some measure be due
o the short time permissible in the model for the flow of water up
he estuary. Probably the Author would reply that the model had
een carefully calibrated before the introduction of the obstruction
o as to give a correct relation between the tidal levels in the model
nd those in nature, but even then the introduction of obstructions
f varying magnitude might possibly introduce some unknown factor
ffecting the time-scale.

Such criticism as he had made was not intended in any way to
elittle the value of the Paper. Despite the limitations that he had
mentioned, it had now been recognized, he thought, by all harbour
nd river engineers that model-experiments were an essential pre-
iminary to any operations on a large scale that might affect the
egime of a tidal river or estuary ; scale models were also becoming
a recognized means for the preliminary study of improvement-
works in non-tidal rivers.

Mr. R. D. GWYTHER wished to refer to the distortion of scale in Mr. Gwyther.
models. In the case of a barrage on the river Tigris, with which
his own firm had been concerned, a number of model-tests had been
carried out to ascertain the effect of constructional operations on
the bed of the river during flood. The model had a horizontal-
scale ratio of 1 : 250, a vertical-scale ratio of 1 : 60, and a time-scale
ratio of 1 : 32. The distortion was approximately $4\frac{1}{4}$, as against
37 in the case of the Humber and Dee models and $42\frac{1}{2}$ in the case
of the Severn estuary. Those scales had been arrived at from
practical considerations, one being to obtain turbulent flow at the
correct proportional velocity, and another being to form the river-
bed slopes in the model correctly without exceeding the angle of
repose of the material. The actual side slope in the river-bed was
generally not greater than 1 : 4, and the sand of the river-bed would
stand under water at a slope of 1 : 1, so that a distortion of $4\frac{1}{4}$ was
reasonable. The model was approximately 42 feet long, roughly
one-half of the length being occupied by stilling-pools and baffles
for the supply of water, which was controlled so that the discharge
was always correct for a given river-level. The bed in the model
was made of river-sand and the banks of clay.

A test was made of the model by moulding the river-bed to repre-
sent conditions previous to a flood of the river Tigris in December,
1936 ; the rise and fall of the river was then reproduced and the
resulting changes in the river-bed measured. The agreement
between the changes shown by the model and the recorded changes

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Mr. Gwyther. in the river-bed was remarkably close. During the experiment was also confirmed that scour was caused by the increased velocit of flow which accompanied a rapid rise of the river, and it was estimated that that increase in velocity amounted to about 40 per cent. of the normal velocity of the river at the particular height That figure was of especial interest in cases when it was desired t calculate discharges on a rapidly-rising river.

Several other experiments had been carried out on the model; point of particular interest were that the model had been constructed at the site of the works, that the results obtained from the model without the facilities and refinements of the laboratory agreed very close with what was actually found to have taken place in the river, and that the model had been initiated and operated by the contractor.

Dr. Chatley. Dr. HERBERT CHATLEY remarked that the results given in the Paper were really startling. The problem as put by the Author might be turned the other way round, with surprising effects; for example, if an obstruction were removed from an estuary, then instead of getting higher high waters and lower low waters (as all the old text-books and general experience would seem to show), the reverse would occur. He had not the slightest doubt about the accuracy of the Author's results, and regarded the Paper as a most important contribution to river-engineering, because it showed that there were certain dangers in the rules of thumb to which it was so often necessary to work in ordinary practice.

The results given were so remarkable that some peculiar cause was to be expected. The Author himself had suggested that a resonance effect was responsible. If that were so—as might well be the case—then it would differ in different estuaries according to their shapes. In the case of the Severn, the estuary was peculiarly bluntly diverted at Frampton, and a reflected wave might be set up.

The Author did not refer to the effect of the convergence of the banks. In the case of the symmetrical model, the convergence was such that throughout the length of the model the wave increased in range in the unobstructed case, and it was not unnatural that an obstruction should modify the convergence of the stream and thereby modify the wave. In the actual estuary of the Severn it would be difficult to define the convergence, but no doubt the Author would agree that the presence of obstructions did modify the effective convergence, and, as there was one particular convergence within which the tidal wave did not change in range, a change of convergence would necessarily alter the tidal range. If the convergence were greater, the tidal range would increase; if the convergence were less, it would decrease. It would be very interesting to hear the Author's opinion on that point.

The change in the surge-time caused by an obstruction seemed to

Dr. Chatley.

e very potent. It appeared that if the surge-time of the wave changed, then there would be a continuous change in the form of the wave. If the periods at Avonmouth and at Frampton were different, then the wave-form between those two places was continually changing, and he did not know which could be considered as the characteristic one for the computation of the tidal energy-loss.

He did not know how far the Author would discriminate between central obstructions of moderately streamlined shape and side obstructions of the nature of groynes. It was conceivable that with a central streamlined shape the energy of the influx would not be greatly modified, so that the velocity between the piers would be increased and scour would occur.

With regard to the Severn, the peculiar formation of the Shoots would presumably affect the results, but it was not represented in the Author's symmetrical model. The section there was so peculiar that, at least at the lower stages of the tide, a large part of the section was not functioning at all but was simply dead water, so that the question of the convergence and divergence of the stream at different levels again arose.

Mr. Binnie.

Mr. W. J. E. BINNIE, Vice-President, remarked that in the case of the large model of the Severn it would be seen from p. 211 that an obstruction of 63·6 per cent. of the waterway caused a lowering of high water of spring tides at Gloucester of 16 inches; the horizontal scale of that model being 3 : 8,500. In the case of the smaller model, however, with a horizontal scale of 1 : 40,000, it appeared from the Table on p. 216 that, with an obstruction of the waterway of 60·0 instead of 63·6, the lowering of high water of spring tides at Gloucester was only 7 inches; what was the cause of the apparent discrepancy?

Mr. Carpmael.

Mr. RAYMOND CARPMAEL remarked that on p. 217 the Author stated: " . . . at the Stones the piers rest on a rock plateau so that the question of erosion . . . does not arise." He could assure the Author that erosion could and did occur in the marl which overlay the Severn tunnel and outcropped over a considerable area of the English Stones.

In his Paper [1] on the cementation of the Severn tunnel, he had explained that a number of pipes or chimneys through the marl above the tunnel had been discovered by the cementation; in certain cases—one in particular—the cement had reached the bed of the river and formed a large mushroom protecting the marl. He could assure the Author that not only did erosion of the unprotected marl take place, but also of concrete, either in mass or in bags, placed in position as additional protection to the river-bed.

The Author also stated on p. 217 : " To prevent complications due to any erosion of the bed altering the cross-sectional area, it was

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[1] R. Carpmael, "Cementation in the Severn Tunnel." Minutes of Proceedings Inst. C.E., vol. 234 (1931–32, Part 2), p. 277.

Mr. Carpmael.   stabilized for the purpose of these experiments by a light wash of weak cement." It would appear that unless the whole of the bed of the Severn in the neighbourhood of any obstruction were coated with concrete, erosion would proceed and the conditions would become different from those in which the results were obtained in the model the bed of which had been stabilized. It seemed that the deductions made from the model might therefore be misleading.

Mr. Du Cane.   Mr. C. G. DU CANE understood from the Author's conclusions that the volume of tidal flow in an estuary might be actually increased by an obstruction. That phenomenon was of great interest, and was contrary to what had in the past been accepted theory. It was a little difficult to believe that it happened, because it was known that in some cases at least the exactly opposite process of clearing the channel in an estuary produced a greater tidal flow; in view of that apparent paradox he would like to have a little more proof before finally accepting the view put forward.

The Author suggested that the phenomenon might be due to partial resonance. Mr. Du Cane had no doubt that that might reasonably account for the higher high water shown by the model; but was there really an increase in the total volume of tidal flow? He took it that by " partial resonance " the Author meant a form of surge or wave. It seemed possible that in measuring the higher high water in the obstructed model the Author was measuring the crest of that surge, and that other parts of the estuary might show a diminution of level at the same time, so that, in spite of the higher high water, the total volume of flow might be less than without the obstruction. Perhaps if the Author would continue his experiments and make simultaneous observations of the water-level at various points above the obstruction, that point might be cleared up; the subject was of very great interest, and well deserved further study.

Mr. Borer.   Mr. OSCAR BORER said he appreciated that the Author had had to stabilize the bed of his model, because otherwise there would be no finality at all in the experiments. The river Ouse, with which he was concerned, had such an erodible bed that scour occurred wherever a structure was erected in it, whereas in model work, as in all other investigations, it was necessary to work from some fixed basis. None the less, it did not follow that models were not to be relied on, and he had been surprised to hear it suggested that a certain experiment relating to a hydraulic structure in the Fens had given a false result. Naturally, if a model were used it had first of all to be proved—it was not just a plaything by means of which anyone who did not understand what he was doing could expect to obtain a result. With a model of the upper reaches of the Ouse it was now possible to carry out investigations on all the sluices and locks, and it was found that the model gave reliable and accurate results. The

hoice of bed-material was not critical, as had been proved both by Mr. Borer.
Inglish and by American experience, but whether it was possible
o rely on a model in a quantitative or only a qualitative sense
aaturally depended on the scales that were used. In the models of
he upper reaches of the Ouse natural scales were employed, and the
esults in every case were almost exact ; that had also been the
experience at the Berlin and Delft hydraulic laboratories.

In the construction of a bridge across the Holland Deeps at
Meerdijk, which was about 2 kilometres wide, it had been found
possible to introduce an obstruction of at least 30 per cent. with no
apparent effect on the river above. That confirmed the Author's
observation that a fairly big obstruction in an estuary would have
very little effect on the tidal rise and fall.

Velocities of flow appeared from the Paper to be reproduced
accurately by models, even on a scale-ratio of 1 : 42. Their accuracy
was demonstrated by the exact correlation of the tidal curves
obtained from the model with those in nature.

Mr. H. W. S. HUSBANDS remarked that in the models mentioned Mr. Husbands.
in the Paper the ratio between the horizontal and vertical scales
ranged from 1/25 to 1/100. Had the Author come to any conclusion
as to what limit should be set to that distortion ? Obviously if a
natural scale could be used it would be better to do so.

The choice of the horizontal and vertical scales was evidently
governed by practical considerations and by the necessity of avoiding
excessive distortion, but he understood that the only way to deter-
mine the time-scale was to find by trial a value that gave a reasonable
reproduction of the natural phenomena. Had the Author been
able to find a method of working it out mathematically ?

Mr. G. J. GRIFFITHS had been greatly impressed by the extra- Mr. Griffiths.
ordinary accuracy of the small model which the Author had used.
He would be interested to know how the Author arrived at the bed-
contours for such a small model, as they would presumably govern
to a very large extent the accuracy of the results. He hoped that
the Author and others would extend their investigations so as to
determine the actual cause of the increase in the high-water level
caused by an obstruction. He was very familiar with the estuaries
of the Severn, Mersey, and Dee, and therefore appreciated the value
of tidal models for the study of the conditions brought about by
changes in channels and sandbanks, which were frequent.

*** Mr. ERNEST BATCHELOR observed that in describing the Mr. Batchelor.
experiments on a small model of the Severn estuary (pp. 213 and 214),
the Author had not stated what modifications would result from

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*** This and the following contributions were submitted in
writing.—SEC. INST. C.E,

Mr. Batchelor. making allowances for the flow of river-water into the estuary unless that flow were relatively small it might have affected the calibration of the model, particularly in the upper part of the estuary.

The Paper was of great value in indicating possible effects of the construction of a Thames barrage, now forming the subject of an Inquiry by the Port of London Authority. It was proposed by the Thames Barrage Association to construct the barrage at the head of Gallions Reach at Woolwich. The water would be maintained at a level near to Trinity High Water, which was 12·53 feet above Ordnance datum. The barrage would have locks to permit the passage of vessels, and sluice-openings of about one-third of the cross-section of the river would be provided. It was suggested that the barrage would be of value for increasing general amenities and facilities for sport, for preventing flooding, and for providing a reserve of drinking water in case of damage by bombing to pumping-stations and water-mains. In one experiment by the Author the estuary had been blocked at Hock Cliff, where the Severn widened rapidly; that point was about 13 miles above English Stones, where the estuary again rapidly widened (p. 226). Conditions in that experiment were perhaps sufficiently similar to those of the barrage at Woolwich to warrant conclusions being drawn applicable to the barrage. The blocking of the estuary resulted in a lowering of high-water level at Frampton, just below Hock Cliff, from 12·0 inches to 4·8 inches, a fall of 7·2 inches. Hence it might be concluded that the Thames barrage would reduce high-water level in the Thames at and below Woolwich. Occasionally a large storm-surge in the North Sea, due to very strong winds from the N.W. to N. over the major part of the North Sea, was superimposed on the astronomical tidal wave. When the maxima of the storm-surge and astronomical tide were almost simultaneous, and the tide was a spring tide, the effects in London were serious. That had been the case at about midnight on the 6th–7th January, 1928, when the flooding caused great loss of life and damage to property. As a result the flood-defences had been raised about 6 inches. Moreover, much larger disturbances of the tidal wave due to storm-surges had been detected by an examination of the tide-curves at Southend since 1911, the largest being one of as much as 11½ feet on the 31st December, 1921, whereas that of 1928 was only 5 feet. Fortunately the maximum of the former occurred near tidal low water. Were a surge of 11½ feet superimposed on high water of a spring tide at Southend the result in London would be appalling, as many square miles would be flooded, including Westminster. No reason had been assigned why the maxima of large surges and tides had not synchronized in the past, and till some explanation had been found, such a disaster, though improbable, should be recognized as possible. The Report of the Committee

appointed after the flood of 1928 to consider the question of floods from the river Thames [1] stated that it had been estimated that the cost of the addition of a foot or more to the height of the existing defence-walls for the whole 45-mile length of the river-front within the County area might well amount to millions of pounds.   The rise in the water-level at Gallions due to the surge of the 6th–7th January, 1928, was 5 inches greater than at Southend, high-water level being 17 feet 7 inches above O.D.[2] ; with a surge of $11\frac{1}{2}$ feet at Southend the rise would have been, proportionately, $11\frac{1}{2}$ inches greater.   Had the maxima of that surge and the tidal wave been simultaneous, the level of high water at Woolwich would then have been 24 feet $7\frac{1}{2}$ inches above O.D., about 7 feet $\frac{1}{2}$ inch higher than in January, 1928.   The barrage would probably reduce that level to less than 24 feet above O.D.   The barrage could be designed to keep water back up to any desired level ; if the defence against surges which would be provided by the barrage were supplemented by a short wall to the high ground to the south, and by works of a length of about $1\frac{1}{2}$ mile to the north as far as the Northern Outfall Sewer, and a small work to the north of that, full protection of London against the appalling losses consequent on a surge as high as the maximum hitherto recorded at Southend could be provided at a cost but a small fraction of that of raising the river-walls.

The sluice-area of the proposed barrage would be about one-third of the cross-section of the river.   *Figs. 4, 5* and *6* in the Paper showed that the tide-curves above the barrage, with the sluices open, would not be materially different from those before its con-struction.   The form of the tide-curves on the 6th–7th January, 1928, was very similar to the normal, the only noteworthy abnormal feature being the excessive height above high water.   The maximum natural flow hitherto recorded in the Thames at Teddington in 24 hours was 3,260 million cubic feet on the 18th November, 1894, that being about twice the ordinary maximum natural flow.   Increasing that proportionately to the increase in the catchment-area, the maximum flow at Woolwich would have been about 4,000 million cubic feet in 24 hours, or 2,000 million cubic feet in 12 hours.   The volume of water in the Thames between Teddington weir and the site of the proposed barrage, between L.W.O.S.T. and the level of high water near midnight on the 6th–7th January, 1928, was, in round figures, 3,000 million cubic feet.   That volume had been discharged during the ebb, which lasted 7 hours 35 minutes.   It was clear that if, at a time of exceptional flow over the weir at Teddington, the lake above the barrage were emptied during the ebb, and if the flood-water were allowed to accumulate during the flood-tide, the level of the water in the lake above the barrage could be kept well below the

[1] Cmd. 3045.   [2] *Ibid.*, p. 12.

Mr. Batchelor. maximum yet attained.　Even, therefore, if the maxima of a storm-
surge equal to the greatest yet observed at Southend and of a spring
tide were simultaneous and coincided with a flood in the Thames at
Teddington not a little greater than the maximum flood yet recorded,
the barrage would enable the flood-level in the Thames above Wool-
wich to be kept well below the crest of the existing defence-works.

Dr.
Cunningham.
　　Dr. BRYSSON CUNNINGHAM remarked that the unexpected and
undoubtedly startling conclusions arrived at in the Author's valuable
Paper were not only at variance with all preconceived ideas on the
subject, but they involved a paradox which, it seemed to him, was
scarcely adequately explained.　On p. 231, it was stated that " in
estuaries of the type and proportions examined, a constriction of
section up to a certain point increases the level of high water at all
points above the constriction and increases the volume entering and
leaving."　From that it would appear that (within the limits
indicated) the smaller the sectional area of entrance, the greater the
volume of water entering the estuary.　The obvious inference was
that the velocity of influx would be correspondingly increased.
That increase in velocity might be brought about in part by the
augmented head due to the penned-up water at the entrance, but
he ventured to suggest that it would be insufficient to produce the
observed result, and that there might be another and important
source of augmentation in a conceivable reflex action from the
surface of the obstructed area producing a local wave rising above the
normal surface-level of the tidal inflow.　He would be glad if the
Author would say whether such a reflected wave was observable
in his experiments, and whether in his opinion it exercised any
influence on the rate and amount of inflow.　The subject of estuarial
obstructions to tidal and river flow was so important from the point
of view of navigation that full explanation should be sought for the
Author's apparently anomalous results.　Until that was done, there
was bound to be scepticism about the applicability of the conclusions
to actual estuaries (as distinct from models), and in his opinion they
could only be accepted with considerable reserve.

The Author. 　　The AUTHOR, in reply, said that two speakers had referred to the
phenomenon, mentioned in the last paragraph of the Paper, that at
Shoreham the widening and deepening of the harbour-mouth had
resulted in depressing the rise of spring tides from 18 feet to 16 feet.
He had come across the information only within the last 3 months.
It was of interest as being one of the few records of cases where the
widening and deepening of a harbour-entrance had lowered the level
of high water, and it was in agreement with experiments which had
recently been conducted on the effect of training-walls in the Dee
below Connah's Quay, where it had been found that, starting with
training-walls 600 feet apart and gradually reducing the gap to

00 feet, a maximum height of high water at Connah's Quay was The Author.
btained when the width was about 300 feet.

In that connexion he would also draw attention to experiments
arried out on the large model of the Severn for the Barrage Com-
nittee, which were recorded in Appendix E of the Report.[1] Those
particular experiments were carried out to determine the effect on
ides and current-velocities of the obstruction caused at various
tages of the construction of the barrage at the English Stones.
The experiments showed that when the deep channel of the Shoots
vas filled in, the high-water level at spring tides above the barrage
vas higher than with the Shoots open. The following levels of high
nd low water were taken from pp. 285 and 295 of the Report :—

	High-water level : feet above O.D.	
	Shoots open.	Shoots blocked.
Avonmouth	21·8	21.7
Beachley	23·3	23·2
Sharpness	24·0	24·5
Framilode	25·2	25·6
Gloucester	23·5	24·3

It had been suggested that the general phenomenon of an obstruc-
tion causing a rise of high-water level might be due to causes other
than resonance, but he had found it difficult to visualize any other
cause, whereas the resonance explanation would appear to fit in with
all the observed facts. The tidal period was longer than the natural
period of oscillation of the column upstream, and anything that
lengthened that natural period brought the two periods closer
together and would thereby tend, other things being equal, to increase
the amplitude of the oscillation, and therefore to increase the level of
high water and to reduce the level of low water. On the other hand,
the introduction of an obstruction introduced a loss of energy which
would tend to reduce the magnitude of the oscillation. The behaviour
of the tides in any particular estuary was the resultant of those
effects, and it was not possible to deduce any general law which
would apply to a series of estuaries ; each one had to be dealt with
as a separate problem. For example, the Humber estuary was very
different in shape, especially in its upper reaches, from that of the
Dee or of the Severn, and the results of obstructions in the Humber
were very different from what they were in the Dee and the Severn.
It was interesting to note that in the experiments on the symmetrical
estuary (pp. 223–226), reducing the mean depth of water increased the
tidal range by increasing the natural period of oscillation and thus
bringing the periods of natural and forced oscillation nearer together.

[1] A. H. Gibson, "Construction and Operation of a Tidal Model of the
Severn Estuary," p. 282. H.M. Stationery Office, London, 1933.

The Author. The idea that the effect might conceivably be due to reflexion had occurred to the Author at an early stage of the experiments, but observation showed no sign of that. If there were such reflexion effects, they would be shown in the form of the tide-curves. The experiments cited above, in which the obstruction in the Shoots was entirely below water-level and in which there could be no reflexion would also indicate that reflexion was not an important factor, if indeed it was a factor. Moreover, even if the results were affected by reflected waves, those would be equally experienced in the estuary itself.

In reply to Mr. Elsden, the tide-curves had not been subjected to harmonic analysis, and in view of the possible errors of observation on the ebb it was doubtful whether anything very useful would emerge from such an analysis. He would, however, examine the results from that point of view.

The calibration of all the models had been carried out with river-flow. Experiments had been carried out both with and without river-flow in all the larger models, but without any effect on the general results. It was true, of course, that where the effect of an obstruction was to increase the level of high water that might well be very objectionable from the point of view of flooding in the upper estuary.

In reply to Mr. Gedye, the original model of the Severn estuary showed that the proposed barrage increased slightly the height of high water at Avonmouth, and also at New Passage at the English Stones. Experiments were carried out on the effect of running with all the sluices closed. The results, given on p. 100 of the Report,[1] showed that that raised the high-water level at Avonmouth by 0·2 foot.

Regarding the suggestion that the introduction of an obstruction in some way altered the time-scale, that was definitely not the case. The time-scale depended only upon the relationship between the horizontal and vertical scales of the model, and no modification which did not change that ratio could possibly affect the time-scale. In each of the models used by the Author, the time-scale had been computed from the well-known relationship $\dfrac{t}{T} = \dfrac{l}{L}\sqrt{\dfrac{H}{h}}$, where t, l, and h denoted respectively time, length and height in the model and T, L, and H the corresponding dimensions in nature.

He had been interested in Mr. Gwyther's remarks about the Tigris model. Its designers had been fortunate in being able to use such a small distortion of scale as $4\frac{1}{4} : 1$. In the case of tidal-estuary models, simply because of the large area which they were bound to cover, it was necessary to have a fairly large distortion of scale in order to obtain turbulent motion. In cases of river or estuary flow where movement of the bed-material was involved, a moderate distortion

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[1] *Loc. cit.*

The Author.

f scale appeared to be rather an advantage than otherwise, whilst
vhere only tidal levels and current-velocities were involved there was
vidence in the present tests that vertical exaggerations of 100 : 1
vere capable of giving results which agreed with the prototype.   In
he models with which he was concerned the bed-contours were
noulded to cross sections taken from the latest available charts.

He appreciated Mr. Borer's remarks about the accuracy of the
esults obtained from the models of the Ouse sluices, and the evidence
is to the small effect on the tidal rise and fall of a considerable
obstruction in the river at Meerdijk.

Dr. Chatley suggested that the presence of an obstruction
affected the convergence of the stream, and that, since in an
estuary with convergent banks the degree of convergence affected
the tidal range, that might be a partial cause of the effect noticed.
However, an obstruction such as a row of bridge-piers only caused
a convergence of flow in its immediate vicinity, which was followed
by an immediate divergence, so that it was difficult to see how
that factor in itself could operate as suggested.   The change in
the period of the natural surge caused by an obstruction was
accompanied by a change in the form of the tidal curve, as
indicated by the difference between the full-line and broken curves
of *Figs. 6* and *8*.   In computing the loss of energy at any section
caused by the obstruction, the two tide-curves obtained at that
section with and without the obstruction were used.   It was perhaps
appropriate to mention that the idea of resonance as affecting the
tidal range in an estuary was not new.   The whole theory of
resonance in estuaries both with parallel and converging banks had
been fully treated for the ideal case of frictionless channels.[1]   It
was by no means essential, in order to have a larger tidal range
at the upper end of a channel than at the seaward end, that the
channel should have convergent banks, as was so often assumed.
Dr. Chatley also asked about the effect of the shape of obstructions.
Some experiments had been done in that connexion, and it had been
found that with a 60-per-cent. side obstruction the same results
would not be obtained as with a 60-per-cent. obstruction in the form
of bridge-piers.   The effect was of the same kind, but not of the same
magnitude.

Mr. Binnie had pointed out that whereas on the smaller model a
60-per-cent. obstruction gave a 7-inch fall in high water at Gloucester,
a 63·6-per-cent. obstruction on the larger model gave a 16-inch fall.
That was so, but it would be noticed that both those points lay on the
smooth experimental curve of *Fig. 4*, and he thought that those two
results formed a rather interesting confirmation of the way in which
the two models, though of very different scales, gave identical results.

[1] H. Lamb, "Hydrodynamics," 4th ed., pp. 258, 267.   Cambridge, 1916.

The Author.   The effect of the larger obstruction had since been tried on the smaller
model, where it gave a fall of 15·5 inches.

He accepted Mr. Carpmael's statement that the English Stones
did erode, but his point was that in a model of the kind in question
for an investigation concerned solely with the effect of obstructions
on tidal levels, it was desirable to prevent complications caused by
any erosion of the bed.

Mr. Du Cane asked whether the volume of flow up and down the
estuary was increased. He had not taken simultaneous tide-curves,
but had obtained tide-curves at different parts of the estuary, and
those could be correlated with reference to a given datum-time, so
that in effect they were equivalent to the curves for which Mr. Du
Cane was asking; those curves did show that the volume was
actually increased. The increase was not great, and the effect would,
of course, depend upon the particular estuary.

In reply to Mr. Batchelor, the calibration of the model was carried
out with river-flow corresponding to mean-flow conditions. Mr.
Batchelor's remarks on the proposed Thames barrage were of much
interest, but the data given in the Paper regarding the effect of an
obstruction at Hock Cliff were not directly applicable to the case of
the Thames. The experiments in which the estuary was blocked at
Hock Cliff had been carried out to determine the relative effect of a
given obstruction (50 per cent.) at Beachley on what were in effect
two different estuaries—one open up to Gloucester and the other
terminating at Hock Cliff.

Since receiving Mr. Batchelor's communication, he had examined
on the Severn model the effect of blocking the estuary at Hock Cliff
without any obstruction downstream. The result was to raise high
water level at Frampton somewhat.

He agreed with those speakers who referred to the caution with
which quantitative observations of scour and erosion in hydraulic
models should be treated. Anyone, however, who had been in
touch with the model work being done at the Waterways Experiment
Station at Vicksburg, Mississippi, was bound to have been impressed
by the very close agreement which had there been obtained between
model results and those of corresponding works in the river in many
schemes of training works.

In conclusion, he thanked all those who had contributed to the
discussion. He was keeping the Severn model in commission in the
Engineering Department at Manchester University, and would wel-
come a visit from any interested engineer who might care to observe
for himself the phenomena outlined in the Paper.

*** The Correspondence on the foregoing Paper will be published
in the Institution Journal for October 1938.—SEC. INST. C.E.

ORDINARY MEETING.

15 February, 1938.

WILLIAM JAMES EAMES BINNIE, Vice-President,
in the Chair.

The following Paper was submitted for discussion, and, on the motion of the Chairman, the thanks of The Institution were accorded to the Authors.

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" The Deformation and Fracture of Metals."†

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INTRODUCTION.

THE essential analytical basis of every type of engineering design is the classical theory of elasticity, which, as normally employed, affords mathematical concepts of the relations of strain and stress in an elastic isotropic solid. The general object of rational engineering design, as applied to structures and machines exposed to atmospheric temperature-conditions—to take a case devoid of the complicating effects of high temperature and corrosive environment—is to produce an effective and economical result in which neither undue deformation nor fracture will occur under assumed loadings and service conditions. Yet, with regard to these statements, reflection will reveal the admitted fact that elastic theory cannot possibly even predict the phenomenon of plastic deformation ; neither can it, of itself, afford any qualitative or quantitative information regarding fracture : it is for this and other reasons that engineering design makes such demands upon the flair, general knowledge and hard-won experience of the engineer, often involving, incidentally, an

† Correspondence on this Paper can be accepted until the 15th June, 1938.
—SEC. INST. C.E.

undue practice of " trial-and-error " methods. Provided that the pressure of the need for improved design can be met by the use of existing materials of known properties, the lack of basic fundamental knowledge may not prove an embarrassment to technical progress ; but the advent of new materials or the requirements of more severe service brings about a sharp realization of the entirely empirical nature of the knowledge of the commonly-observed characteristics of engineering materials. The position is that even the simpler problems of the solid state of matter are baffling in the extreme, in spite of the advances of modern science and of the marked special attention that has been devoted to these problems throughout the world, particularly so during the past 20 years. A most interesting international survey of scientific knowledge and current opinion of the problems of the solid state was made in 1934 in a series of papers [1] at the International Conference on Physics. All metals, and many other engineering materials, are essentially crystalline in their structure, and physical methods have enabled the nature and dimensions of these structures to be determined with the greatest accuracy : there can be little doubt that this crystalline structure is basically responsible for the technical properties with which engineers are so closely concerned. In the first place, however, a much deeper understanding is required of inter-atomic and inter-molecular forces : despite the great contributions to knowledge made by atomic physicists, the theoretical strength of pure iron, for example, is not known. Nevertheless, from associated physical characteristics, the indirect inference appears to be generally accepted that the commonly-observed strengths of metals are much lower than would be expected, if this property depended entirely upon the perfect geometrical structure theoretically assigned to the crystal-line state. To account for this suspected discrepancy, and especially to explain the property of plasticity—an unpredicted and most surprising property of metals—a number of theories have been evolved, which suggest that these characteristics are due to some form of modification or imperfection superimposed on or existing in the crystalline structure. Regarding the correctness of these theories, which suggest the existence of secondary structures, such as the " superstructure " of Zwicky,[2] the " lineages " of Buerger,[3] the

[1] Inter. Conf. Phys., vol. ii, " The Solid State of Matter." Cambridge, 1935.
[2] F. Zwicky, " Imperfections of Crystals." Proc. Nat. Sci., vol. 15 (1929), p. 253.
—— " Why Crystals Exist." *Ibid.*, vol. 17 (1931), p. 524.
—— " Secondary Structure of Crystals." *Phys. Rev.*, vol. 40 (1932), p. 63.
—— " Plasticity of Crystals." *Ibid.*, vol. 43 (1933), p. 765.
[3] M. J. Buerger, " The Lineage Structure of Crystals." *Zeit. für Krist.*, vol. 89 (1934), p. 195.

" block-structures " of Smekal,[1] or the conception of surface or internal flaws originally due to Griffith [2] and made the subject of later experiments by Joffe,[3] Orowan [4] and others, there is no agreement of opinion as to their validity : while, more recently, interesting suggestions ascribing the mechanism of plastic deformation to the presence and effect of migratory " dislocations " have been advanced by Taylor,[5] Polanyi [6] and Orowan.[7] It is not possible here to describe these theories in detail ; a summarized account has been rendered elsewhere.[8] They are mentioned to emphasize the point that plasticity and fracture, which are so frequently encountered as to become commonplace, really represent fundamental problems for which is required the exercise of a considerable amount of ingenuity even to afford a plausible explanation.

Nature of the Problem.

The foregoing discussion may be said to be somewhat remote from the field of practical engineering problems. Even taking full recognition of the general interest which engineers have always shown in scientific problems outside their immediate professional interests, the accusation would undoubtedly be true if it could be shown that their immediate practical problems are capable of satisfactory explanation without the need for an understanding, partial or complete, of the fundamental aspects of plasticity and fracture previously discussed. It is worth while to examine briefly whether or not this latter contention can bear scrutiny. Provision against collapse or fracture under static loading can be avoided if the proper use is made of analytical methods and the experimentally-determined breaking strengths of materials of construction are fully considered in the design stage in relation to the applied loading-conditions : similarly, allowance can also be made for permanent deformations exceeding permissible amounts. Even if failures of this sort occur, a fundamental knowledge of the properties of the solid state is not

[1] A. Smekal, Handbuch der Physik. 2nd edition, Berlin, 1933.

[2] A. A. Griffith, " The Phenomena of Rupture and Flow in Solids." Phil. Trans. Roy. Soc. A., vol. 221 (1921), p. 163.

[3] A. F. Joffe, " Physics of Crystals." New York, 1928.

[4] E. Orowan, " Tensile Strength of Mica." Zeit. für Physik, vol. 82 (1933), p. 235.

[5] G. I. Taylor, " Faults in a Material which Yields to Shear Stress while Retaining its Volume Elasticity." Proc. Roy. Soc. A., vol. cxlv (1934), p. 1.

[6] M. Polanyi, " Lattice-Distortion and Plastic Flow." Zeit. für Physik, vol. 89 (1934), p. 660.

[7] E. Orowan, " Plasticity of Crystals." Zeit. für Physik, vol. 89 (1934), p. 605.

[8] H. J. Gough and W. A. Wood, " Strength of Metals in the Light of Modern Physics." Journal Roy. Aero. Soc., vol. xl (1936), p. 586.

necessary to achieve improved design. What a different picture is presented, however, if fatigue-failure is to be understood and avoided! Estimates from a number of industrial countries agree in assessing the proportion of service failures due to fatigue at as high a value as 90 per cent. of the whole, in spite of the most active and comprehensive attention that has been given to so many aspects of the subject in many laboratories, as shown by the extent of the relevant literature. Whilst many of these failures are due to ignorance or lack of application of existing data, a consideration of the simplest characteristics of that insidious form of failure which occurs by the agency of a creeping and often invisible crack, shows that here are encountered mysterious and unexpected agencies at work for whose mechanism an explanation must be found at the earliest possible opportunity in the interests of rational design, even if only to afford a "background" of understanding against which present and future problems may be considered and overcome.

The simplest facts concerning fatigue-failure are most mysterious. How is it that a material with ultimate tensile and compressive strengths denoted by a and b respectively, can be completely fractured in an apparently entirely "brittle" manner by the application of a cycle of stress having the stress-limits of na and mb, where n and m may have values as low as from $\frac{1}{4}$ to $\frac{1}{2}$? Further, why does fracture occur in this case only after the application of what may be a very large number of repetitions of the cycle, and why is the number directly related to the numerical value of the applied stress-range? What are the inner progressive changes taking place in the material that produce such a fracture? Is "time" or "number of stress cycles" an operating factor? Even to these simple questions no purely mechanical study, however extensive, of fatigue, as providing merely test-data, has ever provided an approach to a satisfactory explanation. Under the auspices of the Department of Scientific and Industrial Research, a most extensive series of related researches has been in progress at the National Physical Laboratory, extending over a number of years. For the purpose of present discussion, these researches have fallen into two groups.

The researches in the first group have been directed to the solution of a wide variety of immediate problems undertaken to provide data directly applicable to design, and they appear to have been successful in achieving their object. Among the major aspects dealt with, some of which are still in progress, may be mentioned :— effect of type of stressing ; impressed conditions ; understressing and overstressing ; influence of mean stress and of temperature ; cyclical stress–strain relations ; effect of surface-conditions and of stress-concentrations due to changes of section ; fatigue-resistance of full-

sized components; investigation of accelerated methods of determining fatigue-range; effect of test environment and corrosion-fatigue; resistance to combined fatigue-stresses; fatigue of welded joints; frettage-corrosion; impact-fatigue. The results of such work have been published in a number of publications too large for detailed reference here: summarized surveys [1, 2, 3, 4, 5] have been prepared from time to time.

It was fully recognized, however, that the purely mechanical methods of attack adopted in that class of investigation could throw no light on the basic problems of fatigue and rupture. In 1921, therefore, work was commenced on a second series of researches specifically directed towards the study of the characteristics of deformation and fracture of metals, especially by fatigue, with regard to the inner crystalline structure. It may be stated at once that it was always considered that, as far as the crystalline structure was concerned, the mechanism of plastic deformation and fracture must be the same under all types of stressing; particular attention has always been devoted to fatigue-stressing, as possessing two inestimable and unique advantages. Firstly, by a proper choice of stressing-system, complete fracture can be caused without involving any appreciable change in the external dimensions of the test-piece; and secondly, the test can be interrupted at frequent intervals so that progressive stages of the deformation and impending rupture can be successively studied without involving any alteration in the externally-applied stressing-system.

In the first stages of the work, the use of the metallurgical microscope was added to the available mechanical methods. An extensive study was made [6] of the changes in the microstructure of polished specimens of various ductile metals, in the usual form of crystalline aggregates, subjected to safe and unsafe ranges of various types of stress-cycle. It was definitely established that, with every metal investigated and under every stress-system employed, slip-bands were formed under safe and unsafe ranges alike, the only difference

[1] H. J. Gough, " Fatigue of Metals." London, 1924.

[2] ____ " Fatigue Phenomena, with Special Reference to Single Crystals." Cantor Lectures. Journal Roy. Soc. Arts, vol. lxxvi (1927–28), p. 1045.

[3] ____ " Corrosion-Fatigue of Metals." 11th Autumn Lecture. Journal Inst. Metals, vol. xlix (1932), p. 17.

[4] H. J. Gough, H. L. Cox, and D. G. Sopwith, " Design of Crane Hooks and other Components of Lifting Gear." Proc. Inst. Mech. E., vol. 128 (1934), p. 253.

[5] H. J. Gough and H. V. Pollard, " The Strength of Metals under Combined Alternating Stresses." Proc. Inst. Mech. E., vol. 131 (1935), p. 3.
____ " The Effect of Specimen Form on the Resistance of Metals to Combined Alternating Stresses." Ibid., vol. 132 (1936), p. 549.

[6] H. J. Gough and D. Hanson, " The Behaviour of Metals Subjected to Repeated Stresses." Proc. Roy. Soc. A., vol. civ (1923), p. 538.

being that in the former case slip ceased at a certain stage of the experiment, while in the latter case fatigue-cracks developed in regions of heavy slip. It was thus established that metals could be cold-worked under cyclic stresses, without fracturing, in exactly the same manner as under static stresses, slip being essentially a harden-ing process, but the observations suggested that each material possessed some definite limit of strain-hardening which, if exceeded locally, led in some manner to the formation of a progressive crack. The paths of the slip-bands indicated unmistakeably a general relation to the crystalline structure, for the detailed examination of which crystalline aggregates were entirely unsuitable. It seemed necessary to establish the planes and direction of slip, also to deter-mine the part played by the crystal-boundaries. At that time (1923) extremely large single crystals, prepared by artificial laboratory methods, were becoming available, presenting a means of attack upon metals in their simplest form and devoid of the complicating effect of the boundary. More important still, by means of X-ray analysis, an exact correlation could be made between the slip- and twin bands produced, as observed using the metallurgical microscope, and the induced stressing-system, using analytical methods. The opportunity for obtaining this much greater insight into the phenomena of plastic deformation and cracking was at once taken up and vigorously pursued. The behaviour of single crystals of aluminium, iron, zinc, copper, antimony, bismuth, and silver was extensively studied in a series of researches which have been sum-marized elsewhere.[1] Some of the principal conclusions can be briefly stated. Deformation by slip always occurred on definite types of crystallographic planes and on definite crystallographic directions in those planes, the actual plane and direction being controlled by the very simple criterion of the greatest component of resolved shear stress. As in the case of the aggregates, such slip occurs under safe and unsafe ranges alike, fatigue-cracking occurring in areas of heavy plastic deformation. A comparative study of the behaviour of single crystals and specimens containing a small number of large crystals showed that the boundary, as such, exerted no appreciable influence on the deformation and fracture of the individual crystals; hence these can be safely taken as properties of the crystalline structure itself : the really surprising fact has been established [2] that, in specimens containing six crystals and subjected

[1] Eighth Marburg Lecture, by H. J. Gough, " Crystalline Structure in Relation to Failure of Metals—Especially by Fatigue." Proc. A.S.T.M., vol. 33 (part 2), (1933), p. 53.

[2] H. J. Gough and G. Forrest, " A Study of the Fatigue Characteristics of Three Aluminium Specimens Each Containing from Four to Six Large Crystals." Journal Inst. Metals, vol. lxviii, no. 1 (1936), p. 97.

to cyclic stressing, the slip-band distribution and fatigue-cracking conformed closely to a stress-analysis based only on the external loading and the crystal-orientation of the individual crystal, and without reference to the adjoining crystals or the intervening boundaries. Probably one of the most significant observations revealed that after the visible production of slip-bands had ceased, a period of cyclic history followed after which the fatigue-crack appeared and spread. This afforded direct confirmation of the previous surmise that the initiation of fracture resulted from the further cyclic straining of a structure which had been initially disturbed by plastic deformation ; or in other words, while a certain amount of slip may have a strengthening effect, a greater amount may lead indirectly to the start of a crack.

These researches had thus provided a great addition to exact knowledge of deformation under fatigue with regard to the geometry of the crystalline structure, but it seemed evident that the actual critical conditions of the structure at fracture still constituted a problem for which more refined physical methods had to be employed if quantitative data were to be obtained.

PREVIOUS WORK.

In relation to the precise knowledge which has been recently obtained and which will be described later, it is of interest to recall that some of the very early work on single crystals, recorded in a Paper [1] published in 1926, gave rise to a tentative suggestion that deformation produced a distorted state of the structure in which adjacent elements were displaced relatively to each other, producing local strains in the lattice, and that the fracture condition was reached when these strains exceeded a certain critical value. This speculation arose from a consideration of the deterioration of the sharpness of the spots obtained in the X-ray analyses. Some years later, experiments made by Taylor and Gough [2] furnished some quantitative data of the conditions of the actual slip-planes of single crystals after the latter had been subjected to static tensile stresses, and also to cyclic stresses. These experiments showed clearly that the crystalline structure became distorted in an exactly similar way by both types of stresses, the nature of the distortion being very much greater along the direction of slip than in the transverse direction.

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[1] H. J. Gough, D. Hanson, and S. J. Wright, " The Behaviour of Single Crystals of Aluminium under Static and Repeated Stresses." Phil. Trans. Roy. Soc. A., vol. 226 (1927), p. 1.

[2] Footnote ([1]), p. 254.

By every indication, both observed and inferred, the elusive cause of the initiation of the fracture of ductile metals thus appeared to be a critical condition of distortion of the structure that was almost certainly sub-microscopic, demanding a further extension of the experimental methods previously employed.

The scope of the inquiry having been narrowed in this manner, it was decided to study the problem by means of the method of X-ray diffraction, which had been greatly developed by physicists during the same period for the study of the solid state. The applicability of the method turns on the fact that by its very nature the phenomenon of X-ray diffraction arises out of the internal crystalline structure of matter, and is a process which is influenced directly and markedly by just those variations in the sub-microscopic structure to which one of the Authors had been led by the line of researches outlined above. Also, at the National Physical Laboratory precision methods had been developed,[1] the X-rays having been applied particularly to the nature, sequence, and influence of the changes occurring in the cold-working of metals, so that it was possible to concentrate at once on the influence of static and cyclic stresses on the inner structure of test-specimens, and to correlate the observations systematically with the process of deformation and fracture. The first phase of this research, in which the Authors became associated, was published in 1936 [2] and related to the behaviour of a normalized mild steel under five different modes of stressing. A further stage of the work has just been completed, and it is mainly with this that they deal in the present Paper.

Before going on to describe these researches, however, it may perhaps be as well to mention some aspects of the X-ray method of approach. It is not necessary to describe the method in detail since this is now well known, having been described, in particular, before The Institution in a recent James Forrest Lecture by Sir William Bragg [3] who, with Professor W. L. Bragg, has pioneered the subject in Great Britain for so many years. The features the Authors

[1] W. A. Wood, " Incidence of Lattice Distortion and Orientation in Cold-rolled Metals." *Phil. Mag.*, seventh series, vol. xiv (1932), p. 656.

—— " The Effect of Lattice Distortion and Fine Grain on the X-Ray Spectra of Metals." *Ibid.*, vol. xv (1933), p. 553.

—— " Lattice Distortion in Nitrided Steels and Theory of Hardness." *Ibid.*, vol. xvi (1933), p. 719.

—— " Examination of Electro-deposited Nickel Coatings by X-Ray Diffraction." *Ibid.*, vol. xx (1935), p. 964.

[2] H. J. Gough and W. A. Wood, " A New Attack upon the Problem of Fatigue in Metals, using X-Ray Methods of Precision." *Proc. Roy. Soc. A.*, vol. 154 (1936), p. 510.

[3] Sir William Bragg, " The Crystal and the Engineer." *Journal Inst. C.E.*, vol. 6 (1936–37), p. 181.

would wish to emphasize are those on which are based the particular applications with which they have been most closely concerned. At the same time they would like to make the point that, although by the X-ray method changes in a metal are not seen directly, but only indirectly, in so far as the changes affect an X-ray spectrum, yet the implications of the spectra may be regarded as definitely established, the process being as real to the X-ray worker as direct visual examination is to a microscopist.

Some Effects of Deformation on the X-ray Diffraction-Spectra of Metals.

As used in engineering construction, a metal is an aggregate of grains possessing the same crystalline structure ; in an ideal metal each grain would be a perfect crystal with the constituent atoms accurately arranged according to a definite spatial pattern on a scale which can be measured. In virtue of the crystallinity a family of planes can be found, each constituted by a layer of atoms in a fixed geometrical pattern and separated by a constant distance d, which is the distance perpendicular to the planes at which the same pattern is next repeated. Various families traverse the crystal, with characteristic spacings, and the important feature, as far as the X-rays are concerned, is that each family will reflect an incident beam of wavelength λ at certain angles given by the Bragg law $2d \sin \theta = n\lambda$, where n is an integer, and θ denotes the angle of incidence and reflexion. The exact details will be found in the works referred to [1,2]. For the present purpose the important point is that it is possible to use any family of atomic planes as a mirror for an X-ray beam, and thus to have available a means of judging the inner perfection or imperfection of the grain on an atomic scale, merely by studying the sharpness and other features of the X-ray reflexions.

Figs. 1 (a) illustrates the simplest case of reflexion by a grain in which the family of planes depicted run parallel from end to end of the grain, as they must in a perfect undistorted crystal. The perfect crystal thus gives a sharp image of the X-ray source (assuming proper experimental conditions), and, if a point source were used, a sharp spot would be recorded on a suitably-placed photographic film. Even if the incident beam were somewhat divergent, as in practice it always is, the reflexion would still be sharp since, according to the Bragg law, the planes automatically select from the beam for reflexion only those rays which make the correct angle θ with the planes.

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[1] Footnote (^{3}), p. 256.
[2] W. L. Bragg, " The Crystalline State." London, 1933.

Figs. 1 (*b*) indicates the change that would occur if the grain were dislocated into two components inclined to one another ; the tilted fragment now makes the angle θ with another bundle of rays not previously reflected, so that two reflexion spots would be recorded. As the grain becomes further broken down, the reflexions would become more numerous and drawn out into a continuously extended reflexion. This is to be expected from the analogy of reflexion by a fragmented mirror ; but, compared with the optical case, there is the limitation imposed by the Bragg law, according to which the reflected beams must always be deviated through the same total angle 2θ fixed by the spacing d of the planes. This confines the

Figs. 1.
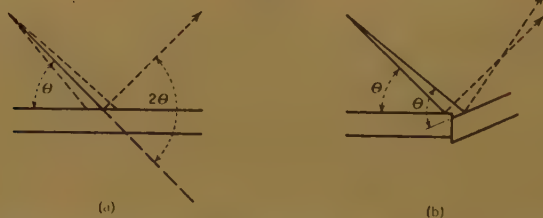
(*a*) The perfect crystal selects from a divergent beam the ray inclined at the reflecting angle θ and gives a sharp reflexion.

(*b*) The broken crystal possesses facets which may be inclined at the angle θ to divergent portions of the beam ; the reflexion is not sharp. (The effect is exaggerated for illustration.)

REFLEXION BY A PERFECT AND BY AN IMPERFECT CRYSTAL.

spread, in space, to the surface of a cone about the direction of the incident beam and with half-angle 2θ, or, on the photographic film, to an arc or ring (Debye ring) where the cone cuts the film. Thus it will be seen how the spreading of the reflexion in this way shows at once the extent to which the grain may be fragmented or broken up and, since, in special cases such as electro-deposited chromium, particles may be as small as 10^{-6} or 10^{-7} centimetre, and still give reasonable reflexions (according to experimental conditions), it will also be seen how the dispersion can be followed beyond the limits of the microscope. If the fragmentation results in an entirely random distribution (with respect to the orientation of the original crystals) of crystal-fragments, then the reflected beam could form a complete ring. This type of change can be magnified by using special techniques,[1] but the above remarks will make the principle clear.

[1] W. A. Wood, " The Variation with Temperature of the Thermal Conductivity and the X-ray Structure of some Micas. II—The X-ray Examination of the Structure." Proc. Roy. Soc. A., vol. 163 (1937), p. 199.

In contrast to the elongation of a reflexion along the diffraction-ring a radial diffusion may arise in certain circumstances and may cause a sharp line to diffuse into a broad band. This would occur, for instance, if the same family of planes in the different reflecting crystal-fragments, owing to local inhomogeneous stresses, were compressed or dilated so that the spacings varied from d to $d \pm \delta d$. Then the reflexions would range over the corresponding range of from θ to $\theta \pm \delta\theta$. The permissible extents to which this effect could occur can be estimated from knowledge of the elastic constants of the metal : at nominal stresses within the observed rupture-strengths, these amounts can only be small.

Radial broadening on a more pronounced scale may also arise when the size of the crystal-fragments becomes very small. The X-ray reflexion from a family of planes is the resultant of the contributions from each of the planes to which the incident beam penetrates. It is the interference of these contributions which confines the reflected beam to the directions imposed by the Bragg law, and the sharpness with which the reflected energy is concentrated in these directions, other things being equal, is simply proportional to the number of co-operating planes, and therefore to the size of the fragment. With customary experimental dispositions, broadening of the line due to this effect begins when the fragment is less than about 10^{-4} centimetre, and thereafter increases in inverse proportion to the linear size, thus providing a valuable and sensitive means of estimating particle-size beyond the limits of microscopy.

Finally, the Authors would refer to a further modification which is concerned more intimately with the structure of fragmented metal when in a condition of severe internal stress. This may be illustrated by *Figs.* 2 (*a*), where a family of reflecting planes intercepts the paper in the parallel lines P. Each plane represents a layer of atoms, and if these were points lying rigidly on the planes, the traces P would simply be lines as drawn. Actually, however, the atoms are positive nuclei at the centre of a characteristic distribution of negative electrons, which in a free atom may be regarded for all practical purposes as being confined to a sphere of action with a definite radius. The formation of the crystal may be considered as a packing of these spheres with minimum energy, fixing the arrangement and spacing of the atomic planes, and the equilibrium of the structure being obtained as the result of the interaction of the positive nuclei and the atmosphere of the negative electrons. Electrons appreciably effective in controlling this equilibrium will be the outer electrons furthest from the over-riding influence of the nucleus. These points were mentioned before passing on because, after all, in any study of deformation and fracture, the two ultimate

factors will always be these forces holding the atoms together, and holding together the crystals or crystal fragments. The influence of the outer electrons on the equilibrium of a structure was implicitly recognized first by Hume-Rothery [1] in work on alloys where, other things being equal, it becomes a factor determining the phase or structure which the associating atoms prefer to take up. The well-known effect of cold-working in changing, or in lowering the temperature required to produce a change in, the phase of an alloy, may be considered a related phenomenon where a preliminary instability of a structure is brought about by distortion. It is thus a matter of particular interest that the X-ray spectra are controlled directly by the distribution of electrons about the atomic planes. This distribution will have a statistical average arrangement which

Figs. 2.

(a)   (b)
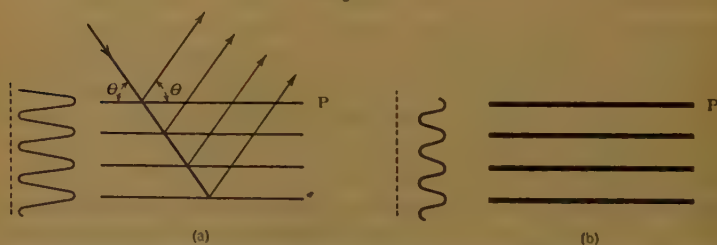
(a) The curves represent (arbitrarily) the mean electron-distribution between the atomic planes P of a perfect crystal.

(b) The curves illustrate the diffusion of the mean electron-distribution which would result from a staggering of the atoms from their correct positions on the planes.

has been represented diagrammatically by the projected curves to the left side of the planes depicted in *Figs. 2 (a)*. It is evident that, in calculating the intensity of a reflexion, formed by adding the components scattered by the electrons, the contributions of the electrons away from the peaks will be out of step with those from the peak itself, and that, *a priori*, the shape of the distribution-curve will therefore affect the intensity of the recorded X-ray reflexions. It will be noted that the positions (values of θ) of the spectra are not changed, since the periodicity of the planes is unaltered; nor is the radial breadth of the lines affected, since this depends on the number of planes only. It is merely that less energy is concentrated in the spectral lines themselves and more is scattered generally into the background of the spectrum. The application to the structure of a crystal-fragment when in a state of internal stress

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[1] W. Hume-Rothery, " The Structure of Metals and Alloys." Inst. Metals Monograph and Report, Series No. 1, 1936.

is illustrated by *Figs. 2 (b)*, where it is supposed that, as a result of local stresses, the atomic planes are distorted and the atoms, instead of being centred on the planes, are slightly staggered permanently about their mean positions. The projected electron-distribution possesses peaks which are now less sharp, as shown at the left of the figure. It has been shown by Hengstenberg [1] that quite a small displacement of this type will affect the spectra quite appreciably : it will, of course, be realized that, assuming the usual stress–strain relations, quite a small change in the interatomic distance could be associated with considerable internal stresses. This brief discussion of the effects of the presence of internal strains in the crystalline structure on the efficiency of diffraction of the X-ray beam is of vital importance in connexion with the results of the recent experimental work to be described.

Now in the deformation of metals the ideal grain is of less concern than the disturbances produced by external stresses, and it will be sufficiently clear, without going into any further detail, that in the X-ray method there is a means of showing up such disturbances in a more fundamental domain than previously has been accessible. It is hardly necessary to add that much requires still to be done in technique and in interpretation. The main principles, however, are clear, and it may be claimed that the following work, by utilizing the method, has permitted for the first time a really full and systematic survey of the process of deformation and the conditions at fracture of an industrially-important material under both static and fatigue-conditions.

DEFORMATION AND FRACTURE OF A MILD STEEL AS OBSERVED BY X-RAY DIFFRACTION.

A brief summary will first be presented of the first stage of the work (1934–35), which dealt with the fracture of mild steel in the normalized condition, so as to be able better to explain the logical extension to the later work which is described in more detail.

Fracture under Static Loading.

The first case was of specimens deformed under progressive static loading until fracture took place. Especial care was exercised to start with material which initially was free from distortion and in which the individual grains were in as near approach to the perfect state as can be obtained in practical crystals. This involved excep-tional care in the preparation of specimens, and in etching away the

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[1] J. Hengstenberg, " Lattice-distortion in light metals." *Zeit. für Elek-trochem.*, vol. 37 (1931), p. 524.

machined surface, so that the surface exposed to the X-ray beam consisted entirely of perfect, unbroken grains, a point of some importance since previously most X-ray work had been on metals in which the structure of the grains was initially in an uncertain condition. The X-ray reflexions then gave lines or rings (actually the experimental disposition was such as to secure a complete ring from a characteristic family of planes) which consisted of sharp separate spots, each spot being a reflexion from a grain in the volume penetrated by the X-ray beam. In any but an exceptionally fine-grained metal this spotted type of ring is, with usual experimental arrangements, characteristic of the normalized, stress-free, state of the metal: an example is shown in *Figs. 3* (*a*) (facing p. 266).

Specimens subjected to increasing static tensile loading and examined at intervals gave the following results: up to the limit of proportionality, which for this steel was 14 tons per square inch, no alteration in the nature of the reflexions took place, but at that point occasional reflexion-spots showed evidence of elongation along the diffraction-ring in the manner described, thereby indicating the dislocation of the corresponding grains into smaller fragments with varying inclinations. It is desired to make it quite clear that, at every stage, the process of fragmentation was of a dual nature, resulting in (*a*) a number of relatively large portions of crystal, whose orientation did not differ very greatly from that of the original grain, giving a " dislocated grain " and, simultaneously, (*b*) a very much greater number of fragments of a very small and definite size, possessing random orientation, to which the specific term " crystal-lites " is applied for the reasons mentioned below. At the yield-point, at a stress of 16 tons per square inch, this effect extended suddenly and markedly to all the grains. On further loading the effects became still more pronounced, especially in respect of the number of widely-oriented crystallites into which the initial grains were disintegrated. Then, when the fracture-stage was reached, there was a further rapid increase in the formation of crystallites until at fracture the specimen behaved to the X-ray beam as a medium of randomly-oriented crystallites in which all traces of the original grains were submerged. Without going into details, which are set out in the original Paper,[1] the Authors would like to emphasize two important points.

The first is concerned with the size of the crystallites. From the formation of the continuous diffraction-ring and from the radial broadening, it is possible to show that the fragmentation of the grains, originally about 10^{-2} centimetre in size, produces crystallites of size 10^{-4} to 10^{-5} centimetre right from the onset of permanent deforma-

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[1] Footnote ([2]), p. 256.

tion ; at no stage, however, does the fragmentation result in measurable further disruption to a smaller size (not even in such a process as cold-rolling, when extreme deformation can be produced, are the crystallites broken down indefinitely ; [1] in this connexion, it must be remembered that the X-ray method is particularly sensitive to changes in particle-size between 10^{-4} and 10^{-7} centimetre). This most interesting process of fragmentation, shown so clearly and definitely by X-ray evidence, suggests the original presence in the crystals of some form of intrinsic weakness, permitting with reasonable ease a dispersion of the structure into the crystallites, which then resist further fragmentation much more strongly ; other observations of an entirely different nature have led to a similar conclusion, resulting in various theories to which previous reference has been made.

The second point is the wide divergence in orientations of the crystallites produced. It has been recognized from X-ray work [2] on non-metallic crystals, such as rock-salt and calcite, that large crystals may consist of mosaics of crystallites, but the orientation of these blocks does not differ greatly from that of the parent crystal ; the deviations in the metallic crystallites as now observed, which may exist and still preserve coherence, are entirely of a different order. This is a special characteristic of the metallic state, overlooked in many current theories of plastic deformation, and is probably responsible for the fact that the possible plastic deformation of a metal is also of a totally different order from that exhibited by other crystalline materials (except for occasional very special cases). Similar experiments were made using static compression and torsional stress, with identical results. All these experiments revealed the characteristics referred to above and showed that the fracture-stage was that at which the fragmentation of grains into a distorted crystallite formation had proceeded to the extreme. It was as if the metal accommodated itself to the overstrain by means of this dissociation of the grains, and that the fracture started where the dislocation was locally complete and the accommodation was utilized to the full.

Fracture under Fatigue Loading.

The case of specimens fractured by fatigue may now be considered. The essential difference between the fracture produced by static and cyclic stressing is that, in the first process, failure occurs as the ultimate effect of a progressively increased stress, whereas, in the

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[1] Footnote (^{1}), p. 256.
[2] Footnote (^{2}), p. 257.

second, failure occurs as a result of many repeated applications of
the same cycle of stress, even though either stress-limit of this cycle
is much less than the value required to produce fracture by a single
application. It was, therefore, of great interest to examine the
structure of representative specimens subjected to cycles of stress,
on the one hand just less than the limiting range of stress, and, on
the other just exceeding this range, and to make the examination at
regular intervals during the history of the specimens. Also, by
choosing various values of the mean stress of the cycle it was possible
to produce fracture under conditions (a) where the upper stress-
limit was less than the static yield-point and, hence, the deformation
of the specimen as a whole was negligible, and (b) where the upper
stress-limit exceeded the yield-point and the specimen, as a whole,
deformed considerably under safe and unsafe ranges alike. In this
way, the effects of plastic deformation and true fatigue were studied
separately and independently. Various types of cycles of both
torsional and direct stresses were investigated in this way. The
results showed that after allowance had been made for the effects of
the application of the superior stress of the cycle, the repeated
application of a safe range of stress produced no further change in
the crystalline structure of the test-specimen. If, however, the stress-
cycle were such as to lead to fatigue-failure, then the behaviour was
quite different. The specimen exhibited a progressive fragmentation
of the crystalline grains which was, essentially, exactly the same in
kind as that produced by the static deformation and fracture
previously discussed. The degree and rate of production of the
breakdown depended on the extent by which the applied stress-
range was in excess of the limiting fatigue-range : as this excess
increased, so the rate of deterioration of the structure also increased,
and thus the limiting condition of fracture was attained in a smaller
and smaller number of cycles. There was thus made available, and
for the first time, a physical means of differentiating between the
effects of safe and unsafe ranges of stress, and, moreover, a demon-
stration of a physical process which accounted entirely for the
familiar form of the usual curve of stress-range *versus* number of
repetitions. In the region of the fatigue-failure, the structure was
broken down just as much as in the specimens fractured under static
load, the X-ray photographs showing a completely continuous,
diffuse diffraction-ring. This breakdown was greater than any
which would be accounted for by the local deformation caused by
the crack itself, since the experiments were arrested before the
fatigue-crack had attained any appreciable size.

Now although it is evident that the fracture-stage is associated
with the fragmentation occurring in the manner indicated, it does

not follow that the failure is due to this effect. The experiments on
the static deformation showed that an increase in load produced
further breakdown of the grains until equilibrium with the added
load was established, so that the fragmentation is actually intimately
connected with the effect of strengthening or strain-hardening. The
evidence is rather that fracture occurs when the strengthening due
to this process is locally exceeded, and in this respect attention may
be drawn to one difference between fracture under static and fatigue
failure, namely that, in the former, practically the whole of the
material appears to arrive simultaneously, or nearly so, at the
extreme condition of fragmentation, whereas, in the latter, that
condition is localized, leading to the formation of a relatively small
number of fatigue cracks. Nevertheless at each such local point of
failure, the structure was in an exactly similar condition to that
broken under static loading.

Further Experiments on Cold-Worked Material.

Although these experiments had thus provided so much new and
consistent evidence, the Authors were not satisfied that the fracture-
stage represented merely a state of fragmentation ; consideration
of the well-known increased strength and hardness conferred on
ductile metals by cold-working processes is sufficient to raise some
doubt on this point : also, certain observations made in the experi-
ments suggested that, at the fracture-stage, severe internal stresses
were also present. It was therefore considered that further light
would be directly thrown on this very important aspect of the
problem by the investigation of a mild steel similar in composition
to that already studied but possessing initially a structure which
already had been reduced to the fragmented condition, a condition
which is to be obtained by preliminary cold-working of the material.
A supply of normalized mild steel was therefore obtained, some of
which was cold-worked by rolling so that a reduction in cross-
sectional area of 49 per cent. resulted. The structure of this cold-
rolled material was found to be entirely fragmented in the sense
explained above, and was therefore particularly suitable for the
extension of the investigation since the characteristics of deformation
and fracture were unlikely to be confused by further fragmentation.
The opportunity was also taken, in this second phase of the work, to
make a further study of the behaviour of the steel in the normalized
state but utilizing a technique which had been improved in the
interim so as to permit examination of identical grains through
successive stages of an experiment.
Throughout these further experiments, the fatigue method of

testing was used, and, of the various types of cyclic stressing available, choice fell on that of reversed direct stresses since, by this method, specimens of the present material can be fractured without undergoing any appreciable change in external shape or dimensions. As a consequence the effects observed may be regarded as being due almost entirely to fatigue-action alone, and as being free from the complications which would be introduced by a superposed resultant permanent deformation.

Experimental Data.

Before considering the results the principal experimental data may be recorded.

Material.—The mild steel (N.P.L. reference-mark JPH) had the following percentage composition : carbon 0·12, silicon 0·22, manganese 0·62, sulphur 0·008, phosphorus 0·018, nickel 0·06, chromium trace. This was stated by the makers to have been normalized at 900° C., whilst metallurgical examination at the N.P.L. showed that the structure was typical of a low-carbon steel in the hot-rolled condition. The cold-worked material was obtained by cold-rolling the normalized material from an original diameter of $\frac{7}{8}$ inch to a diameter of $\frac{5}{8}$ inch, giving a reduction in area of 49 per cent.

Static Tensile Properties.—(1) *Normalized Steel* : Upper yield-stress, 18·0 tons per square inch ; lower yield-stress, 17·2 tons per square inch ; ultimate tensile strength, 28·2 tons per square inch ; breaking stress (on final area), 68 tons per square inch ; elongation at fracture (on 4-inch gauge-length), $33\frac{1}{2}$ per cent. ; reduction of area at fracture, 72 per cent.

(2) *Cold-Rolled Steel* : No yield-point ; ultimate tensile strength, 56·6 tons per square inch ; breaking stress (on final area), 73 tons per square inch ; elongation at fracture (on 3-inch gauge-length), 5 per cent. ; reduction of area at fracture, 51 per cent.

Fatigue Endurance-Tests.—For each condition of the steel a series of endurance-tests was first carried out to establish, in the usual way, the limiting range of stress above which fracture of a specimen took place after a certain number of cycles but below which a specimen was safe after an indefinitely large number of cycles. These tests were made without interruption. Then, the critical range having thus been determined, further specimens of each type were tested at stress ranges slightly greater and, also, slightly less than this range, each test being interrupted at regular intervals, usually after 0, 10^3, 10^4, 10^5, 10^6 . . . . . cycles, when an X-ray examination was made in order to permit of a study of the progressive changes in structure which might have occurred. The slight periods of rest

Figs. 3.

(a)

(b)

Initial condition.

After 10⁷ stress-cycles.

X-RAY DIFFRACTION-RINGS OF SPECIMEN 2A11, TESTED AT A SAFE
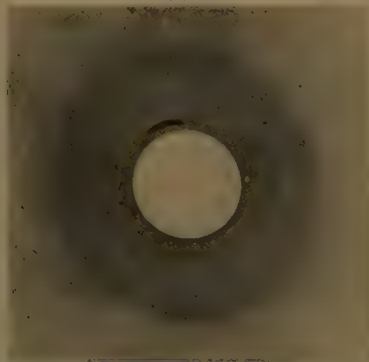STRESS-RANGE OF ±11½ TONS PER SQUARE INCH.

Figs. 4.

(a)

(b)

Initial condition.

After 10³ stress-cycles.

(c)

(d)

After 3·03 × 10⁷ stress-cycles : away from fracture.   After 3·03 × 10⁷ stress-cycles : at fracture.
X-RAY DIFFRACTION-RINGS OF SPECIMEN 2A12, TESTED AT AN UNSAFE
STRESS-RANGE OF ±12 TONS PER SQUARE INCH.

5168.

Figs. 5.

(a)

(b)
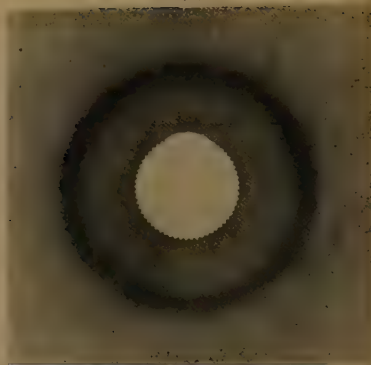
Initial condition.

After $1 \cdot 1 \times 10^7$ stress-cycles.

X-RAY DIFFRACTION-RINGS OF SPECIMEN 1A8, TESTED AT A SAFE
STRESS-RANGE OF ±19½ TONS PER SQUARE INCH.

Figs. 6.

(a)

(b)
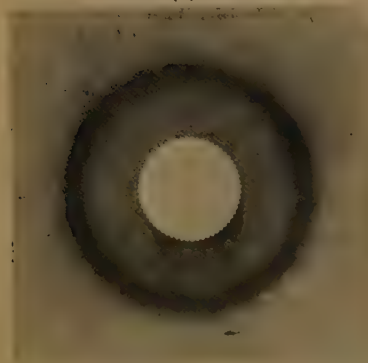
Initial condition.
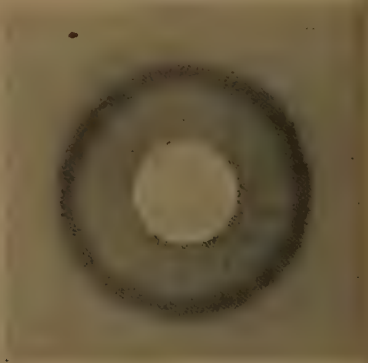
After 10^3 stress-cycles.

(c)

After 3×10^6 stress-cycles.

X-RAY DIFFRACTION-RINGS OF SPECIMEN 1A10, TESTED AT AN UNSAFE
STRESS-RANGE OF ±20½ TONS PER SQUARE INCH.

involved by interruption of the tests were found to result in a slightly increased fatigue-limit, but such an effect was immaterial to the object of the research, although revealing the interesting property of " recovery " for future detailed examination.

The limiting fatigue-range of the normalized material was found to be clearly defined at $\pm 11 \cdot 6$ tons per square inch and that of the cold-worked material at $\pm 19 \cdot 7$ tons per square inch : the relevant stress–endurance curves, plotted to logarithmic co-ordinates, are given as *Fig. 8* (facing p. 268).

It is unnecessary to describe in detail the results of all the tests and examinations that were made. The behaviour of a few typical specimens will reveal the principal new facts disclosed by the research.

(A) *Material in the Normalized Condition.*—A specimen (reference 2A11) was subjected to a safe stress-range of $\pm 11\frac{1}{2}$ tons per square inch and was examined after 0, 10^3, 10^4, 10^5, 10^6 and 10^7 cycles had been applied. In the initial state the specimen gave a diffraction-ring consisting entirely of sharp reflexion-spots, *Figs. 3 (a)*, showing that the material consisted entirely of large perfect grains. Every subsequent photograph gave similar diffraction-rings which were exactly identical, spot for spot as shown in *Figs. 3 (b)*. In view of the large number of reflexion-spots involved, this result, using the later technique, thus showed with a degree of sensitivity not hitherto employed that the repeated stress-cycles are entirely without appreciable effect on the crystalline structure ; progressive deterioration is entirely absent and the specimen can safely withstand an unlimited number of applied stress-cycles.

Another specimen (reference 2A12) was tested at a stress-range of ± 12 tons per square inch (a range slightly exceeding the fatigue-range) and was examined after 0, 10^3, 10^4, 10^5, 10^6, 2×10^6, 3×10^6, 4×10^6, 10^7 and at $3 \cdot 03 \times 10^7$ cycles, when fracture occurred. The initial photograph, *Figs. 4 (a)*, gave the sharp separate reflexion-spots characteristic of the normalized material. After 10^3 cycles (*Figs. 4 (b)*), however, the X-ray photograph showed a complete change in the identity of the spots recorded, thus indicating at once a disturbance of the crystalline structure of the specimen. Further changes of this type occurred after 10^4 and 10^5 cycles, but thereafter slowed down to such an extent that the slightest differences only existed until 10^7 cycles, when a further change in identity of the spots took place which was again very marked when examined at fracture (*Figs. 4 (c)* and *(d)*). This test was very informative in indicating that the rate of modification of structure decreased as the test proceeded until a stable state is approached but not quite reached ; the rate of change accelerated during the last stages of test, and fracture resulted.

A specimen (reference 2A15) tested at a stress-range of ± 13 tons per square inch fractured after $5 \cdot 504 \times 10^6$ stress-cycles had been applied. It is unnecessary to go into the changes produced in this case as they were similar to the preceding, except to mention that with the increased stress-range the progressive changes did not slow up to anything like the same extent as in the case of specimen 2A12.

It is known from the previous work that higher stress-ranges would have produced more marked, easily visible, and accelerated changes. The above tests showed that even only just above the limiting range effects of the applied stress-cycles on the crystalline structure could be definitely established. These effects, identified with the disappearance of some reflexions and reappearance of others, are early stages of the dislocation and fragmentation described previously causing relative movements of the grains : they show, however, that when the limiting range is only just exceeded the fragmentation must be confined to occasional grains.

(B) *Material Tested in the Cold-Rolled Condition.*—A specimen (reference 1A8) was subjected to a safe stress-range of $\pm 19\frac{1}{2}$ tons per square inch and was examined after 0, 10^3, 10^4, 10^5, 10^6, 10^7 and $1 \cdot 107 \times 10^7$ cycles when the test was discontinued, the specimen being unbroken. The initial photograph, *Figs. 5 (a)*, gave a continuous diffuse diffraction-ring characteristic of a material in the extreme fragmented condition. The photographs at the successive stages of the test showed no noticeable modification of any kind (*Figs. 5 (b)*) so that the cold-worked material appears to obey the same essential criterion for safety as the normalized, namely, absence of progressive change.

A specimen (reference 1A10) was subjected to an unsafe stress-range of $\pm 20\frac{1}{2}$ tons per square inch and was examined after 0, 10^3, 10^4, 10^5, 10^6, 2×10^6, 3×10^6 and $1 \cdot 1374 \times 10^7$ cycles, when fracture occurred. The initial state, *Figs. 6 (a)*, was similar to that of the previous specimen. After 10^3 cycles, however, the X-ray photograph showed a marked drop of intensity relative to the background, *Figs. 6 (b)*. Little change then was noticeable until 10^6 cycles, when a further drop in intensity occurred which was continued at the subsequent stages of test (*Figs. 6 (c)*, and *10*, facing p. 271), until finally the diffraction-ring was only just discernible from the background of the photograph. An examination of the accompanying X-ray photographs will show clearly the progressive changes which occurred in the structure, but such photographs are difficult to reproduce in exactly comparable conditions. It is therefore desirable to record that microphotometer measurements of the magnitude of the changes in intensity were made from the original films. Means were taken

to avoid variations in experimental conditions in obtaining the negatives and in the after-processing. The following quantitative measurements of differences in density between the peak of the diffraction-ring and the background were obtained, which not only reveal the very great final drop in intensity, amounting to 86 per cent., but also indicate, clearly, the slowing-up of the change between the early and later stages of the test.

Number of cycles .	0	10^3	10^4	10^5	10^6	2×10^6	3×10^6
Intensity-difference .	0·21	0·14	0·13	0·13	0·07	0·05	0·03

Thus the effect of the unsafe range of stress is to produce a distinct modification of the structure.

CONCLUSIONS.

The changes were therefore of the nature of those discussed in the final case in the previous section dealing with the effects of deformation on the X-ray spectra ; it was there indicated that if, owing to local stresses, the atomic planes became distorted in a way causing slight but permanent displacements of the atoms from their normal mean positions, then the effect observed above would be produced. The implication of this effect, in connexion with the main problem of fracture, was also discussed in that section, the important point being the bearing of this type of distortion on the equilibrium of structure of the metal.

These experiments thus appear to have established a further important step in the understanding of the problem of fracture. The earlier investigation showed that permanent deformation of the metal produces a peculiar " crystallite-formation " and that a fracture stage becomes imminent when the fragmentation represented by this formation has been carried, at least locally, to the extreme. In addition to this, it would appear, from the later work, that externally-applied stresses are capable in certain conditions of producing severe internal strains and distortion of the crystallites in a manner which may be expected to affect the stability of the atomic structure, and it is in this factor that is seen the possibility of an explanation of ultimate fracture, more fundamental than any that has hitherto been attainable. It may be that the Authors have been fortunate in their selection of material and experimental conditions, and it is certain that much development-work requires to be done, but from further results already in hand, involving studies on other metals and on large single crystals, and from the

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background of other data bearing on the problem, there can be no doubt that the underlying explanation of the deformation and fracture of metals will not be essentially different from the one that they have attempted to present in this Paper.

For one metal at least, and that a metal of considerable importance to engineering construction, the fracture stage, irrespective of the type of applied loading, appears to be a condition of extreme fragmentation into a mass of crystallites of a definite critical size, these crystallites being in a condition of severe internal stress or strain.

A typical example of a fatigue-fracture as seen by the eye is reproduced in *Fig. 7*. Some of the well-known but curious characteristics of fatigue as revealed by mere mechanical tests will be recognized in the simple diagram of *Fig. 8*. Some of the aspects of fatigue, as made available by the metallurgical microscope, are typified by *Fig. 9*, while the condition at fracture, as now disclosed by X-ray methods of precision, is reproduced as *Fig. 10*. The foregoing account of the correlation of these various characteristics, made possible by the researches referred to, will, it is hoped, prove to be of interest to The Institution.

The Paper is accompanied by three sheets of diagrams and fourteen photographs, from which the Figures in the text and the two half-tone page-plates have been prepared.

Fig. 7.
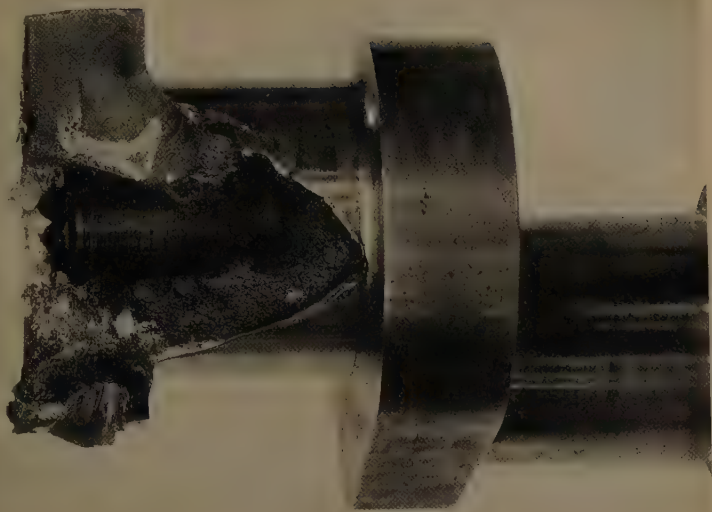
FATIGUE FRACTURE OF ENGINE CRANKSHAFT.

Fig. 8.
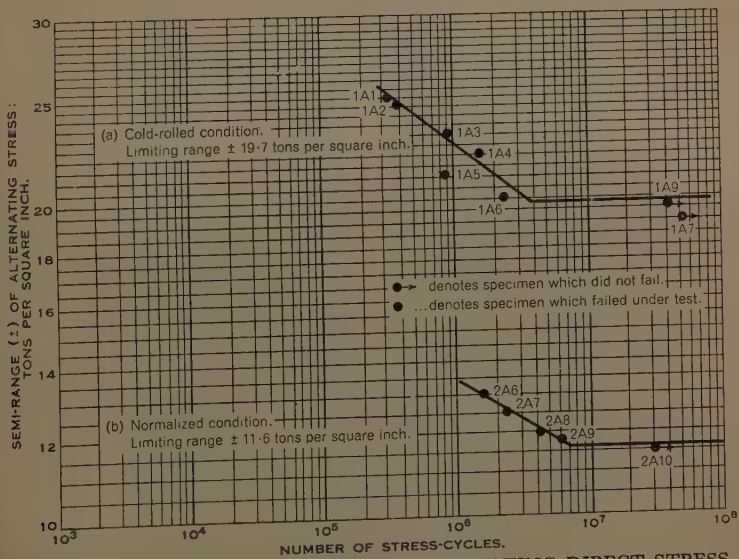
RESULTS OF FATIGUE-TESTS WITH ALTERNATING DIRECT STRESS.

5168.

Fig. 9.

FATIGUE FAILURE OF A SINGLE CRYSTAL OF ZINC, SHOWING SLIP BANDS, TWIN BANDS, AND THE COURSE OF A FATIGUE-CRACK IN RELATION TO CERTAIN CRYSTAL-PLANES.

Fig. 10.

X-RAY DIFFRACTION-RING OF SPECIMEN 1A10, AFTER FRACTURE BY $1 \cdot 1374 \times 10^7$ STRESS-CYCLES OF $\pm 20\frac{1}{2}$ TONS PER SQUARE INCH.

Discussion.

Professor C. E. INGLIS said that the thoughtful and stimulating per which the Authors had submitted was a very fitting sequel to e James Forrest Lecture given by Sir William Bragg at the end of e last session of The Institution. By X-ray analysis of a funda-ental character the Authors had answered many questions relating the deformation and fracture of metals.

It was possible that in certain directions he might be misinter-eting the interesting phenomena recorded in the Paper, but it peared to him that the salient point which emerged was that a ecimen of steel when on the verge of breakdown had its crystal-ains, at any rate in the neighbourhood of the fracture, reduced a disorderly array of very minute particles termed crystallites, ose crystallites being of the order of 1/100,000 inch in size, and such quite beyond the range of microscopic detection. In a rmalized specimen which was tested to destruction by a gradually creasing load, that fragmentation into crystallites progressed pidly as soon as the plastic stage was reached, and, as the load creased from the elastic limit to the ultimate breaking point, the agmentation became more and more complete. From that rogressive replacement of orderly crystals by a disorderly array of ystallites the material seemed to derive increased strength, but ventually a stage was reached when, the fragmentation being omplete, no further assistance could be obtained in that manner, nd the end was at hand. As was pointed out in the Paper, however, at was not the whole story. Whereas a condition of complete agmentation into crystallites appeared to be always the precursor f fracture, the converse (namely, that fracture followed immediately pon a complete state of fragmentation) was not true ; for example, the case of cold-drawn wire the fragmentation might be complete, nd yet the material had a great reserve of strength. In fact, the fe-story of a specimen of cold-drawn wire tested to destruction eemed to begin just where it might be said that death supervened

Professor
Inglis.

in the case of a normalized specimen. The difference between th[
two cases was due to the mysterious phenomenon called wo[
hardening, which was highly developed in cold-drawn material, a[
also developed, though in a lesser degree, in normalized mate[
tested to destruction. Until that phenomenon was explained, [
elastic limit, plastic deformation, and ultimate breakdown wo[
remain unfathomed mysteries. The Authors stated that they h[
as yet no explanation to offer, but he would like to know whet[
any light was being thrown on the problem by X-ray analys[
in particular, had the Authors any positive or negative evide[
available regarding the existence of the " amorphous layer " su[
gested by Sir George Beilby ? In recent years Beilby's theory of t[
amorphous layer had fallen a good deal into disrepute, but he h[
recently come across one or two eminent metallurgists who w[
apparently inclined to revert to it ; they did so, he thought, beca[
that theory, more naturally than any other theory that had be[
presented, made it possible to take into account the way in whi[
elastic limits, plastic deformations, and ultimate stresses w[
dependent upon the rate of loading, which was nowadays found[
be a most important consideration. Crudely expressed, Beilby[
theory of the amorphous layer suggested that when a material w[
subjected to cold working the crystals or crystallites were press[
extraordinarily forcibly against one another ; when they we[
compelled to slide, the abrasion was such that points of conta[
momentarily fused or liquefied, giving rise to a vitreous amorpho[
layer which was probably a good deal stronger than the crystallit[
themselves. The crystallites, being embedded in a matrix of t[
amorphous material, would achieve greater strength, in the sam[
way as glass or concrete could be reinforced by wire nettin[
Perhaps that theory might yet account for the mystery of wor[
hardening.

There was one other point on which he desired to touch. Throug[
the replacement of orderly crystals by a disorderly array of crysta[
lites, material even in its normalized condition seemed to deriv[
additional strength, and that would suggest that for great strengt[
a material was required which was of an amorphous description—[
a solidified fluid which was entirely devoid of any of the planes o[
patterns associated with a crystalline structure. He believed tha[
non-crystalline amorphous materials might soon be produced o[
such extreme strength that the strongest and toughest steel woul[
seem weak in comparison ; synthetic materials of a non-crystallin[
character could already be produced, which on a comparison on th[
basis of weight were much stronger than the toughest steel. Ther[
were many natural products also which could achieve the sam[

ult ; for example, a spider's filament had a tensile strength of Professor
out 70 tons per square inch, but, in so far as it was only one-eighth Inglis.
the weight of steel, on a basis of weight it compared with a
eel which had an ultimate tensile stress of 560 tons per square inch.
gain, considering a synthetic material, glass, if it were not for a
perficial layer which was weak in tension, would certainly have a
sistance of the order of 500 tons per square inch ; even now, by
utting the outside of the glass into an initial state of compression,
that that weakness of the superficial layer was neutralized, a
ughened glass was produced which was comparable in strength
ith a high-grade steel.   It certainly seemed possible that at a date
ot many generations ahead steel might cease to be pre-eminent as a
ructural material, and, just as humanity had passed through the
one and bronze ages, it might even now be nearing the end of the
on and steel age.

Dr. W. H. HATFIELD remarked that he was not quite convinced Dr. Hatfield.
y the new proposals which the Authors had put forward, although
e did not share Professor Inglis's view, for instance, with regard to
he amorphous layer.   He felt that in the light of modern know-
edge it was extremely unlikely that the amorphous phase could
ersist in metals in the cold.   Consideration of that view, however,
d him to a very important point on which he would like to join
ssue with the Authors.   Dr. F. P. Bowden, at Cambridge, had
ecently carried out some very interesting experiments on the
emperatures resulting from the abrading together of two metallic
urfaces, and had clearly demonstrated that even in the case of
teel temperatures were achieved in the neighbourhood of the
melting-point of the metal—that discovery had thrown great light
on the question of seizing.   The occurrence of those high tempera-
ures suggested that the conditions of the Authors' X-ray diffraction
ests might be open to criticism.   He presumed that the specimens
when tested were at rest and taken from the machine.   In that case,
he spectra would not show the atomic arrangements of the material
under the precise condition at which it would fracture, when high
ocal temperatures might exist ; they would actually show the
condition after equalization of temperature.   He would suggest that
probably during the process of fatigue very intense temperatures
were reached in layers only a few atoms thick on the slip planes.
Under the existing pressures, temperatures well below the normal
melting-point might give rise to liquefaction, which on cooling or
reversal of stress would lead to recrystallization. (He did not
consider it possible that the condition of the under-cooled liquid,
which Professor Inglis put forward, would ever arise.)   He would
like to ask the Authors what difference was likely to result in their

Dr. Hatfield. deductions from the important fact that they had studied the specimens after taking them from the machine.

Since he visualized in cold-worked metals such a complex set of conditions, and since he held that the Authors had been studying the ultimate result of the whole reaction, including cooling-down and rest, up to the time at which they had taken the X-ray photograph, he was faced with another difficulty, namely, that it seemed very strange that the crystallites resulting from fragmentation had a uniform standard dimension, apparently of the order of 1/50,000 centimetre. Why should the results of fragmentation ultimately be of the same dimensions? In that connexion, why did the Authors use the term "fragmentation"? It suggested a breaking-up into fragments. If, however, the matter were considered in three dimensions, it would be appreciated that none of the atoms ever got far enough apart during the process of "fragmentation" for there to be any space between the so-called fragments. Would not that process be better described as a re-orientation within the crystal from different centres?

The Authors had cold-worked their mild steel to give a tensile strength of 50 to 60 tons per square inch, but the same steel might be cold-worked to give a tensile strength of 112 tons per square inch. Was it to be assumed on the Authors' theory that when the material was only cold-worked to give 50 to 60 tons per square inch only a certain proportion of the crystals were broken down to the so-called fragment size? Was it to be assumed that the ultimate tensile strength, say 112 tons per square inch, was obtained with complete fragmentation? He was led to ask that question because Dr. Gough in his introductory remarks had mentioned that when a specimen had been broken in torsional fatigue at a stress of $\pm 9\cdot7$ tons per square inch the proportion of crystals fragmented had been found to be 69 per cent. If at fatigue failure under $9\cdot7$ tons per square inch there was a fragmentation of 69 per cent. he did not see how it would be possible to account by fragmentation for the ultimate strength of 112 tons per square inch that might be obtained by cold work.

He was very pleased indeed to see the data given by the Authors regarding the effect of cold work on mild steel. Some time ago he had claimed that the strength was increased in tension as in compression; that had often been debated and challenged, but the Authors had now shown that the safe fatigue-range could be raised from 11 to 19 tons per square inch by cold work.

In the last paragraph of p. 252 the Authors gave a schedule of factors that had to be studied in relation to fatigue-failure; one important factor was, however, omitted. It was necessary to take

to consideration the effect of time under service conditions upon Dr. Hatfield.
the intrinsic properties of the material. It was generally assumed
that if a material were not stressed beyond the elastic range the
atoms still remained in the same positions and the intrinsic properties
of the material were unchanged. He was convinced, however, that
it would be well if engineers in general were to take a great interest
in the intrinsic properties of metals after service, so as to find what
changes, if any, had occurred.

Dr. HUBERT SUTTON observed that the fact—to which the Authors Dr. Sutton.
had called attention elsewhere as well as in their present Paper—that
slip-planes within the crystal were concerned in fatigue-failure had
been of great service in the determination of the causes of failures
of machine-parts in service. The fractured surfaces were often
found to have been damaged by rubbing or hammering after failure,
but it was sometimes possible to secure a section for microscopic
examination right up to the facture, and examinations of that kind
had frequently permitted the mode of failure to be identified as
intra-crystalline, and with an absence of microscopically-detectable
distortion in the structure. The Authors now appeared to have
evolved a very careful technique for studying the mechanism of
failure of parts subjected to various stresses, and their method
appeared to be of special importance since it afforded an approach
to some of the more difficult problems relating to fatigue. Questions
such as the effects of understressing—straining for higher duty by
subjecting the material to lower stresses than those to be withstood
in service—and also the effects of standing, mentioned by Dr.
Hatfield, might be studied thereby.

The Authors had found the size of the particle produced by
fragmentation to be of the order of 10^{-4} or 10^{-5} centimetre ; did
that dimension agree with the dimensions of secondary structures
which had been identified in iron by other methods ?

He believed that in America Dr. C. S. Barrett had been working
in the same field as the Authors, but he did not appear to have been
able to identify such definite stages of the breakdown with his X-ray
pictures[1] as those determined by the Authors. Dr. Barrett appeared
to have observed degrees of plastic deformation both above and
below the endurance-limit, and it would be interesting to know
whether the Authors associated Dr. Barrett's difficulties with the
conditions of stressing, materials, or some other factor in his tests.

In his opinion, work-hardening took place below the fatigue-limit
in fatigue-tests, and he would like to know whether the Authors
believed that the evidence that they had obtained revealed no

[1] *Metals and Alloys*, vol. 8, p. 13. (January, 1937.)

Dr. Sutton. breakdowns into crystallites or such small particles below the fatigue-limit.

In practical experience of the investigation of failures, he had come across many cases of intercrystalline failure; he thought that it was correct to say that in those cases steady stresses had generally been operative, and in many instances corrosion had also been at work. Perhaps the Author's work would provide a means of distinguishing steady-stress failures from variable-stress failures; that was often a difficult practical problem.

Dr. Dorey. Dr. S. F. DOREY remarked that there were certain points in the Paper that were of particular interest to physicists and engineers. In the first place, tensile tests indicated that there was no change in the X-ray diffraction-pattern up to the limit of proportionality. That satisfied him that perhaps, after all, there was a limit of proportionality. Fatigue-tests within the safe range also indicated no change in the structure. Thus it was shown that up to the limit of proportionality and the safe fatigue-range fragmentation of the crystal-structure did not take place, but, when those limits were exceeded, a progressive fragmentation was evident.

Was there any visible characteristic of the final fracture, whether it was tensile or by fatigue, which would satisfy him that some change had occurred in the material? The cup-and-cone fracture produced by a slow tensile test did exhibit a very fine smooth structure, quite different from that of a quick break; a fatigue fracture also showed a very fine structure. To his mind that was evidence of a microscopic or sub-microscopic effect common to the two cases. He therefore looked on fractures, either fatigue or static, as involving a condition of fragmentation. Fracture apparently occurred when strengthening due to a process of strain-hardening was carried locally to the extreme.

In carrying out some experiments on hysteresis-effects in crank-shaft-steels, he had found that above a certain stage, which approximated to the fatigue-limit for reversed stress, the rate of increase in the value of the hysteresis-energy per cycle with stress rapidly increased. The results contained in the Paper indicated that that was also the point where fragmentation of the crystal-structure of the material commenced, and he would like to ask the Authors whether the additional hysteresis per cycle might be due to the process of fragmentation. He had also found that that rapid increase in hysteresis-energy was very much more marked in the case of a ductile steel than of a high-tensile steel, which made him wonder whether the question of internal stress was involved.

With reference to the matters that he had mentioned, and particularly in regard to strain-hardening, he felt that the effect of

stalline size in relation to fragmentation required further study. Dr. Dorey.
e results as they stood indicated that cold-rolled material with a
gmented structure would be the best in service against fatigue-
ading.   In practice, however, forgings were carefully normalized
 remove internal strain due to the effect of working, and were
t into service in a condition of low fatigue-limit.   It was there-
re important to determine what effect the yield-point had.   At
ints of stress-concentration, was crystal-fragmentation due to
tigue prevented once the material yielded ?

Cold-worked material exhibited very much smaller hysteresis-
ergy per cycle than normalized material for the same stress-range.
milarly, in cold-worked material a higher stress-range was required
r crystal-fragmentation to proceed.   That again indicated some
lation between the magnitude of hysteresis-energy and the
ogress of crystal-fragmentation.

It might be of value to study the relation of crack-propagation to
e results of the investigation described.   It was well known that
 the harder steels, whilst cracks were formed at higher stresses
an in ductile steels, the cracks travelled quicker.   Had fragmenta-
on already been produced in the heat-treated condition which
lowed crack-propagation to take place at a much greater rate ?
lternatively, the size of crystals and state of internal stress of the
aterial due to the hardening process might have an important
earing on the rate of failure.   In cases of corrosion-fatigue, was it
ossible that fragmentation allowed the corrosive medium to attack
e particles more completely ?

Dr. A. MÜLLER, referring to the interpretation of the X-ray Dr. Müller.
iffraction-patterns, observed that the Authors clearly realized the
ery complicated nature of the problem.   It was extremely difficult
 say from the X-ray pictures what really happened.   There was
o doubt a breaking-up, but there was one point in that connexion
hich he would like to make.   He had recently been concerned with
n investigation of a similar nature on an unstrained material in
hich an extremely high resolving power had been used, and it
howed that the reflexion-spots did not lie on one ring ; there was
 fairly big spread even in the initial condition on a well-annealed
aterial.   He came rather rashly to the conclusion that that was
ue to a lattice-variation even in the annealed condition, the lattice-
izes of the different crystals differing by an amount varying between
·1 and 0·01 per cent.   Professor W. L. Bragg, however, had recently
ointed out to him that the apparent variation was possibly due to
n optical effect.   Even with a material made up of crystals of
lentical lattice-size, the reflexion-spots would be spread over a
ertain width to start with, thus producing a spread of the ring.

Dr. Müller.   The importance of that optical effect had only recently been realized,
and it would have to be taken into account when interpreting the
width of the ring.

The part played by the atomic forces in determining the mechanical
properties of a material was of interest. A good deal was known
about the forces themselves. For example, Fuchs [1] had recently
calculated the force between two sodium atoms in the sodium
lattice, and had thence predicted the mechanical properties of
sodium. The results had proved to be in remarkable agreement
with those obtained experimentally, so that it was now possible to
predict from first principles what the elastic forces were. The
problem with which the Authors were concerned was, however, far
more complicated. The difficulty might be seen from an analogous
example ; in the case of the planetary motion of two bodies, the
initial conditions being known, it was possible to say exactly what
their orbits would be, but if a third body were introduced the
problem became insoluble. In the case of a metal, there were
millions of atoms, and calculation was hopeless. The central
problem before the Authors at the present moment was to find out
why there were in the metal certain small groups which remained
stable. That was a statistical problem, and had nothing to do
directly with the atomic forces.

Mr. Robertson.   Mr. T. S. ROBERTSON observed that he was particularly inter-
ested in the hysteretic behaviour of metals ; together with Professor
B. P. Haigh, he had examined that phenomenon in mild steels under
fatigue-testing.[2] *Fig. 11* showed the arrangement of the apparatus
they had used. Thermocouples had been arranged around the
specimen in such a way that the hot junctions were all at the middle
and the cold junctions at the ends, so that the effect of any tempera-
ture-drop from end to end of the specimen was completely eliminated.
They had thus been able to measure the temperature-drop from
the centre to the ends. *Figs. 12* showed the results obtained
with two mild-steel specimens, the first large-grained and the second
normalized. The former was of especial interest in the present
discussion. At the beginning of the test there was great primary
hysteresis. Secondary hysteresis followed which was smaller in
amount, and finally, just before fracture, there was considerable
tertiary hysteresis. Those effects agreed very well with the inten-
sity-difference figures of the X-ray diffraction-rings found by the
Authors.

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[1] K. Fuchs, "The Elastic Constants and Specific Heats of the Alkali
Metals." Proc. Roy. Soc. (A), vol. 157 (1936), p. 444.
[2] B. P. Haigh, "Hysteresis in relation to Cohesion and Fatigue." Trans.
Faraday Soc., vol. 24 (1928), p. 125.

Fig. 11.

Figs. 12.

(a)   (b)

STRESS-CYCLES: MILLIONS.
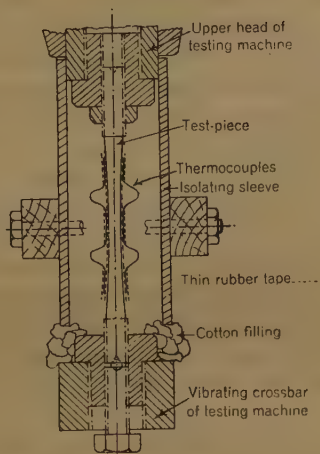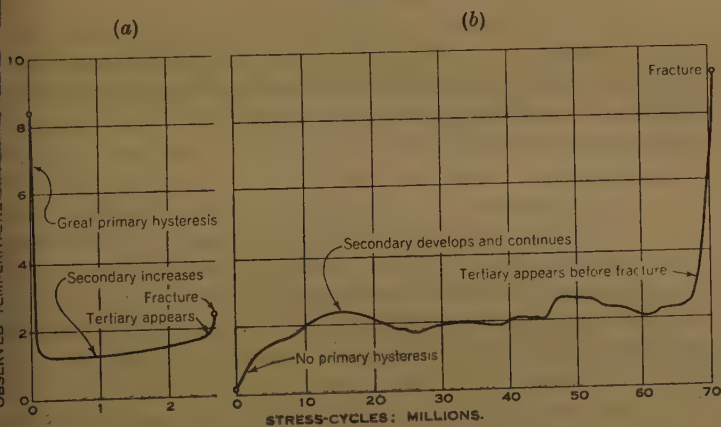
Slowly cooled from high temperature: large-grained.

Tensile strength 31·4 tons per square inch.
Izod value 7·5 foot-lb.

Normalized : fine-grained.
Tensile strength 36·8 tons per square inch.
Izod value 73 foot-lb.

Percentage Analysis (both specimens) :—C 0·27, Mn 0·77, Si 0·19, S 0·034, P 0·012, Ni 0·32.

HYSTERESIS-TESTS AT STRESS-RANGE OF ±15·45 TONS PER SQUARE INCH.

Dr. Orowan.

$*^*_*$ Dr. EGON OROWAN observed that it was very interesting hear that, according to X-ray evidence obtained by the Author there was an upper limit of fragmentation, corresponding to fragment-size of between 10^{-4} and 10^{-5} centimetre. A ver tempting way to explain that observation would be to consider th crystal as built up of blocks of that size, the cohesion between th blocks being smaller than inside them. Such an explanation however, would contradict a great number of facts, so tha another possibility of obtaining a limit to fragment-size had to sought.

There was, indeed, a simple way to understand the existence of fragmentation-limit, which had the particular advantage that it le to a formula for the minimum size of stable fragments. Althoug it contained two quantities of which only the order of magnitud was known, it seemed that that formula might fit well the observ tions. During plastic deformation undoubtedly a great number fragments smaller than 10^{-5} centimetre were formed. In the ca when such a fragment contained severe stresses and distortion the total energy of the specimen would decrease if the fragmer were dissolved by its neighbours by a process of which the macr scopic manifestation was recrystallization. That process could tal place even at temperatures very much below the usual recrystalliz tion-temperature, because the instabilities were much greater in th moment of plastic deformation and the volume affected by th dissolution of a small fragment was much smaller. Thus it had on to be shown that a small fragment could be consumed by its su roundings even if it consisted of an entirely unstressed lattice of th lowest energy possible. To show that, account had only to b taken of the surface-energy of the small fragment. If E denote the free energy per unit volume of the surroundings due to th lattice-distortions produced by cold working, the energy $E \cdot dV$ w gained if the volume of the fragment (the lattice of which might l supposed to be undistorted) increased by dV. At the same tim however, the surface of the fragment increased by dS, and thus i total surface-energy by $\omega \cdot dS$, ω denoting the specific surfac energy of the boundary between the fragment and its surrounding The fragment would be stable if the change of the total energy for virtual increase of its volume vanished; that was to say, if as muc was gained on the volume-energy of cold working as had to b expended on surface-energy, so that $E \cdot dV = \omega \cdot dS$. Assumir that the shape of the fragment was cubic, and denoting the lengt of the cube side by x, its surface was $6x^2$, and the virtual increase

$*^*_*$ This contribution was submitted in writing.—SEC. INST. C.I

he surface-energy was $12\omega \cdot x \cdot dx$. The corresponding increase
f its volume was $3x^2 \cdot dx$, and thus the increase of the total volume-
nergy of the system was $3E \cdot x^2 \cdot dx$. Hence the stability-condition
as $3E \cdot x = 12\omega$. As to the numerical value of ω, it was known
aat the surface-energy of heavy metals against vacuum was of the
rder of magnitude of 1,000 ergs per square centimetre. The surface-
nergy of an internal boundary, however, was much smaller; that
as shown by the low resistance of metals against plastic deforma-
on as compared with their breaking-strength. A probable order of
agnitude of ω was about 100 ergs per square centimetre. On the
her hand, the maximum value of the volume-energy stored up in
ry severely worked mild steel had been determined by Professor
aylor and Mr. H. Quinney to be about 10 calories per cubic centi-
letre. A value of about 1 calorie seemed, however, to correspond
uch better to the conditions of a usual fatigue-test. The formula

$$x = \frac{12\omega}{3 \cdot E}$$

aus gave, with $\omega = 100$ and $E = 1 \cdot 4 \cdot 2 \times 10^7$, $x = 10^{-5}$
entimetre. Allowing for the uncertainty of E and ω as well as of
he observed value of the minimum size of fragments, that accord
as satisfactory.

A more reliable proof of the suggested explanation, however,
ould be to determine the fragmentation-limit at a very low
emperature, such as that of liquid air. At a low temperature the
robability of atomic rearrangements was very small, so that the
maller fragments, although instable, would be prevented from
issolution. Was there any possibility of such an experiment
eing carried out by the Authors?

The AUTHORS, in reply, wished to record their thanks to those
vho had contributed to such an interesting discussion. The dis-
ussion was concerned essentially with certain definite aspects of
he subject, and it would be convenient to deal specifically with
hem.

Work-hardening had been discussed by Professor Inglis, Dr.
Iatfield, and Dr. Sutton. In the Authors' opinion no complete
xplanation of that most important and interesting property of
netals had yet been advanced. Work-hardening might be said to
e evinced by an increased resistance to shearing forces, brought
bout by plastic deformation. The Authors' work showed the effect
f slip on the crystalline structure, resulting in a deteriorated con-
ition consisting of an accumulation of fragmented material in the
orm of crystallites possessing wide variations in orientation.
lthough such a change in the structural condition would be expected

The Authors.
to result in an increased resistance to slip, nevertheless, it appear:
most unlikely that variety of orientation could, in itself, produ
the greatly-increased strength actually observed. Possibly, modifi
cations in electronic distribution, produced by internal stresses,
the formation of nuclei—as in age-hardening—would prove to be t
final explanation. Further, the conditions leading to slip were n
yet known, whilst its actual mechanism remained a comple
mystery. Therefore, whilst the present work might justifiably
said to exhibit many new facts concerning the process of wor
hardening, it did not account for that phenomenon. With rega
to the particular point raised by Dr. Sutton, the Authors wou
recall that their earlier work had shown quite clearly that fragme
tation could result from the application of cycles of safe or unsa
ranges alike; the difference being that in the former case a stab
state was reached, whilst in the latter case the changes were pr
gressive to the stage of actual fracture. Although specially critic
tests had been designed to elucidate that most important aspec
and afforded indisputable evidence of the facts, it was som
what unfortunate that the whole of that work had been ove
looked by Dr. Barrett in some of his writings to which Dr. Sutt
referred.

Various references and inquiries were made by Professor Ing
and Dr. Hatfield with regard to the amorphous state of meta
The Authors felt a strong disinclination to be drawn into anythir
resembling the unfortunate and heated controversies which, in th
past, had raged on that matter. The very beautiful experimen
of the late Sir George Beilby on the nature of polished surfaces ha
never, in the Authors' opinion, been challenged, and they sti
remained a perfect example of excellent technique and undeniab
conclusions. The controversies referred to, however, had arise
from the efforts of over-enthusiastic disciples of Beilby who ha
endeavoured to read into his conclusions much more than the
originator; in so doing they had done considerable harm to h
work. In their Paper, the Authors had endeavoured to prese
solid experimental facts, purposely avoiding excursions into inferen
which the present state of exact knowledge did not justify. The
would, therefore, content themselves by saying that none of the
observations had affirmed or denied the production of any amo
phous material as the result of slip. Nevertheless, they would poi
out that the established lower limit of the crystallite-size indicat
that the major portion of the deformed metal was essential
crystalline.

Two points had been raised, by Dr. Hatfield and Dr. Mülle
concerning the technique employed. There certainly might ha

The Authors.

n some differences between the characteristics exhibited by the
cimens while actually under stress and when the stress was
eased ; also, " recovery " effects by rest might be in operation,
l some experiments on that aspect were in hand.   They had little
ubt, however, that the principal features of deformation and
cture were residual, and were fully revealed by X-ray diffraction-
ctra obtained immediately after the test-specimens had been
moved from the testing machines.   Even employing the most
werful source of X-rays at present available, it would not be possible
take instantaneous photographs, and there seemed little hope of
aining records of extremely transitory phenomena by such means.
e very beautiful experiments of Dr. Müller, using X-ray technique
der conditions of high resolution, were of extreme interest, but,
der the conditions used in the work described in the present
per, the cause of line-broadening to which he referred did not
ply.

everal most interesting questions were asked by Dr. Hatfield,
. Sutton, Dr. Orowan, and Dr. Dorey, concerning the crystallites
ming the fragmented metal.   It was a matter for regret that, at
esent, no physical explanation had been offered for the existence
a lower limiting size for those crystallites : the problem was
racting vigorous research, and evoked constant discussion in
entific circles.   In that connexion, attention might be directed to
e most interesting theoretical explanation suggested by Dr.
owan in his contribution to the discussion (p. 280).   As mentioned
the Paper, the approximate dimension of 10^{-4} centimetre had been
sociated with the results of observations of widely-differing
tures, optical, etching, etc., and also with the " blocks " of the
ggested secondary structures.   Work was in hand on crystallite-
rmation at low temperatures, which, as suggested by Dr. Orowan,
ght throw more light on the process.   The term " re-orientation
thin the crystal " might possibly be used in place of " fragmenta-
n," but, to the Authors, the latter term had the merit of presenting
clearer interpretation of the condition revealed by the X-ray
otographs.   The degree of fragmentation could not affect the
pearance of a fracture, as static and fatigue fractures were essen-
lly different to the eye, although, as the Authors had shown, the
ates of the crystalline structure were identical.

As mentioned by Dr. Dorey and Dr. Robertson, the fatigue-
aracteristics exhibited by X-ray methods and by strain-hysteresis,
o by measurement of heat evolved, were capable of correlation to
nost satisfactory degree.   The " primary hysteresis " corresponded
 the marked initial dislocation and fragmentation : the lesser
secondary hysteresis " was reflected exactly by the slowing-down

The Authors. in the rate of the same processes ; the " tertiary stage" was re]
duced by the final acceleration as fracture was approached, also
some extent by the formation and propagation of the crack.

In conclusion, the Authors would remark that their resear
were being continued, and they fervently hoped that, at some fut
time, they might be privileged to supply experimental data on so
of the fascinating problems yet unexplored, to which such construct
references had been made in the Discussion.

JOINT MEETING WITH THE INSTITUTION OF CHEMICAL ENGINEERS.

18 January, 1938.

SYDNEY BRYAN DONKIN, President Inst. C.E.,
in the Chair, supported by

WILLIAM CULLEN, D.Sc., President I. Chem. E.

" The Treatment and Disposal of Trade Waste Waters."

By ALBERT PARKER, D.Sc., M.I. Chem. E.

(Abridged.)[1]

INTRODUCTION.

NUMEROUS industrial and manufacturing processes of a chemical or chemical engineering nature give rise to waste waters which cannot be discharged in relatively large quantities, except after preliminary treatment, into rivers and streams without causing undue pollution. Satisfactory methods of treatment and disposal of the wastes at little or no cost to the factory are not always easily found, and the problems to be solved frequently require the knowledge and experience of the biologist, the chemist, the engineer and the chemical engineer.

IMPORTANCE OF PREVENTION OF POLLUTION OF WATER-SUPPLIES.

In Great Britain, from 35 to 40 millions of the total population of about 45 millions receive piped supplies of water from undertakings controlled by local authorities, joint boards, statutory and non-statutory companies. The total volume of water supplied by these undertakings is between 1,000 million and 1,500 million gallons per day. Of this quantity nearly 1,000 million gallons, equivalent to about 25 gallons per day per head of the population in the area of supply, are used for domestic purposes ; the remainder is used for various industrial purposes, and much larger quantities are drawn direct from rivers, springs, and wells for agricultural and industrial purposes.

Surface water—from rivers, streams and springs—is the principal source of supply in Great Britain. Many of the surface supplies are

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[1] To be published in full with the discussion in the Transactions of the Institution of Chemical Engineers, vol. 16 (1938).

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obtained from the upper reaches of rivers where there is little or no contamination by polluting discharges. Considerable quantities—several hundred million gallons per day—are, however, drawn by public water-supply undertakings from the lower reaches of rivers below points of discharge of sewage and trade effluents of various kinds, and many undertakings have to incur great expense and shoulder considerable responsibility in purifying the water to ensure that it is fit for general supply.

Available sources of uncontaminated water, both surface and underground, are gradually being allocated; but any probable future demand in this country for water of good quality can be met, provided that determined efforts are made to prevent undue pollution of the resources.

One of the difficulties in preventing or reducing pollution by industrial effluents has been the general apprehension that appreciable improvement in the position cannot be achieved without a serious additional charge on industry. It is true that entirely satisfactory methods of dealing with certain trade effluents, taking costs into account, are unknown. In most cases, however, pollution can be brought down to reasonable limits without serious additional expense. There are in fact circumstances in which the composition of the waste waters can be improved by reducing losses of valuable raw materials, products and by-products in the factory, or the waste waters can be re-used with economy in the manufacturing processes; in some cases products of value can be recovered from the wastes.

LEGAL POSITION.

Disposal by Percolation Underground.—The disposal of waste waters by percolation into the sub-soil is restricted by the obligation not to cause a public nuisance nor to interfere with the rights of other landowners.

Discharge into Rivers.—Disposal of wastes by discharge into rivers and streams is subject to several restrictions. At common law, the pollution of water so as to endanger the health of the public is an indictable offence. Further, every riparian owner has a right to receive the flow of a stream in a condition unaffected by the use made of the stream by other riparian owners; the Courts will protect this right by injunction and damages.

Of all the general Acts on the subject, the most important are the Rivers Pollution Prevention Acts, 1876 and 1893, which deal with both sewage and trade wastes—solid and liquid. It is an offence under these Acts to discharge into a stream any poisonous, noxious, or polluting liquid proceeding from any factory or manufacturing process. Provisions similar to those in the Rivers Pollution Pre-

vention Acts are included in the Salmon and Freshwater Fisheries Act, 1923, in the interests of fisheries.

Discharge into Sewers.—Numerous traders discharge the waste waters from their manufacturing processes into the sewers of the local sanitary authority for treatment in admixture with domestic sewage at the local sewage-disposal works. Frequently, simple preliminary treatment of the wastes at the works of the trader is required before the wastes are admitted to the sewers, and the discharge may be subject to other conditions, including payment.

The extent of a trader's right under the general statutes to discharge wastes into public sewers has not been free from ambiguity. Legal difficulties have in some measure been overcome in certain districts by local Acts of Parliament facilitating the discharge, under controlled conditions, of industrial effluents into the public sewers. The recommendation by the Joint Advisory Committee on River Pollution appointed in 1927 by the Ministers of Health and of Agriculture and Fisheries that the law should be amended on the lines of the several local Acts so as to give traders a right to discharge trade effluents into the public sewers, subject to compliance with regulations and conditions to be made as to payment and other matters, has now been given effect to by the passing of the Public Health (Drainage of Trade Premises) Act, 1937, which comes into full operation on July 1, 1938.

METHOD OF TREATMENT OF TRADE WASTE WATERS.

The most detailed inquiry so far carried out into methods of treatment and disposal of sewage and trade effluents was that of the Royal Commission on Sewage Disposal, which occupied a period of 17 years, from 1898 to 1915.

Among the recommendations of the Commission was the establishment of a central authority to deal with various administrative matters relating to prevention of pollution and equipped to undertake special investigations. No action was taken on the Commission's recommendation. The Water Pollution Research Board, however, was appointed in 1927 by the Department of Scientific and Industrial Research to submit schemes for research on the prevention of the pollution of rivers and other sources of water-supply and on any relevant matters affecting the purity of water-supplies, and to supervise approved investigations. So far as research is concerned, the Board may be regarded as fulfilling the functions of the central authority recommended by the Royal Commission on Sewage Disposal.

In dealing with any problem of disposal of trade wastes, the first important step should always be to consider the practicability of modifying the processes and methods in the factory with the object

of avoiding the production of the wastes or of reducing their quantity and polluting character. There is often the possibility of reducing the losses of raw materials and of valuable products and by-products carried away in the waste waters. Sometimes products of value can be recovered from the wastes, and certain of the waste waters can be re-used in the factory processes.

The principal trade waste waters remaining for disposal, after all practicable steps have been taken to reduce their quantity and polluting character to the minimum, may be divided into three main groups. In the first group there are waste waters, such as those from stone quarries, china clay works, and coal washeries, which contain solid matter in suspension but little or no polluting matter in solution. These wastes can usually be purified sufficiently for discharge into rivers and streams by sedimentation, with or without the addition of chemical coagulants and precipitants, followed in some cases by mechanical filtration. Secondly, there are waste waters such as those from tanneries, cotton bleach works and certain types of paper works, which contain solids in suspension and polluting substances in solution. These require treatment by sedimentation, sometimes with chemical precipitation, followed in many cases by further puri- fication by chemical and biological processes. The third group includes wastes containing polluting substances mainly in solution or colloidal dispersion. Examples of this class are the waste waters from the treatment of iron and steel with acid before galvanizing, from gas-works and by-product coke-works, and from dairies and creameries. Wastes of this kind may require chemical or biological treatment or both.

Waste waters in the second and third groups can frequently be most efficiently treated in admixture with sewage at the local sewage-works, provided that the quantity of trade effluent in relation to the volume of domestic sewage is not too large, and certain conditions, such as discharge of the waste into the sewer at controlled rates of flow and not in flushes, are observed. This method is adopted for the disposal of the effluents from many gas-works.

Applications of some of the principles outlined in the preceding paragraphs can best be illustrated by references to two or three typical examples.

Beet-Sugar Factories.—One example is provided by the results of the investigation carried out by the Water Pollution Research Board on the problem of disposal of effluents from beet-sugar factories.

In a factory of average size, the quantity of fluming and washing water employed is about 3·5 million gallons per day, and the process water usually amounts to about 500,000 gallons per day. Fluming and washing water is as strong in its polluting effect on a stream

as an equal volume of many crude domestic sewages, whilst process water is five or six times as strong. The waste waters from one beet-sugar factory may thus be equivalent, in their polluting effect on a stream, to the domestic sewage from a town with a population of 200,000 to 300,000.

The investigation has shown that the whole or the major quantity of the fluming and washing water, and of the process water, after simple treatment by sedimentation with occasional additions of small quantities of lime, can be re-used in the factory, and that waste waters, necessarily discharged, can be purified to the required extent by biological oxidation in percolating filters.

Milk Effluents.—During recent years, the problems of treatment and disposal of the waste waters from the milk industry have increased in importance with the development of the industry and the establishment of depots and factories each receiving the milk from many farms.

An investigation by the Water Pollution Research Board has shown that even at milk-collecting and distributing depots where the milk is received only for cooling, pasteurizing, and distribution, the waste waters from washing delivery churns, coolers, pasteurizers, tank wagons, other equipment, and the floors usually carry away 0·5 to 1·0 per cent. of the milk received.

From the investigation of the subject it has definitely been concluded that the quantities of polluting matter carried away in the waste washing waters can be considerably reduced by simple and inexpensive modifications in the operations within the factories and by more careful control of the processes of draining and washing. By installing a simple drainage-rack of suitable size and design, the quantity of milk carried away in the water used for washing the delivery churns can be reduced from more than 0·5 per cent. to less than 0·25 per cent. of the milk handled. If adequate drainage-trays were installed at all the depots and factories in this country the total saving of milk would be of the order of 3 million gallons per annum or at least £150,000 per annum.

The next step in the investigation was to study possible methods of purifying the waste waters unavoidably produced. Experiments in the laboratory indicated that it would be practicable to purify the waste waters either by the activated-sludge process or by the process of biological oxidation in percolating filters operated under certain conditions. In the experiments with percolating filters it was observed that, in single filtration, solid fatty matter was deposited in the top layers of the filter which in consequence soon became choked and inoperative. It was found, however, that this difficulty could be overcome by adopting a method of biological filtration

suggested by Mr. H. C. Whitehead and the late Mr. F. R. O'Shaughnessy of the Birmingham, Tame and Rea District Drainage Board. According to this method, the waste waters after sedimentation are passed through two biological filters in series and the order of the two filters in series is periodically changed. Under suitable conditions, treated effluent from the one filter brings about the removal of the solid matter previously deposited in the other filter when that filter occupied the primary position.

With the financial co-operation of the industry, the investigation was extended to include experiments at two large experimental plants. In one plant, the waste waters, after sedimentation, are treated by the activated-sludge process ; in the other plant, the settled waste waters are submitted to biological oxidation in two percolating filters in series, the order of the filters being reversed at intervals of 10 days to 3 weeks. For the activated-sludge plant, the settled crude waste is diluted, if necessary, to give a mixture with a biochemical oxygen-demand of not more than about 50 parts per 100,000 parts. From this mixture, with a period of aeration of 24 hours, a final effluent with a biochemical oxygen-demand of about 1 part per 100,000 is obtained. For the biological filtration plant, the settled wastes are diluted to give a mixture with a biochemical oxygen-demand of not more than 30 parts per 100,000 parts. From this mixture a final effluent with a biochemical oxygen-demand of less than 1 part per 100,000 parts is obtained at a rate of treatment much greater than that usually adopted in the treatment of domestic sewage at sewage-disposal works in this country.

Gas-Works Effluents.—The polluting effluents produced in the manufacture of gas by the carbonization of coal are derived mainly from the aqueous product known as gas liquor or crude ammonia liquor, which is ordinarily distilled for the production of ammonium sulphate, concentrated ammonia liquor, or anhydrous ammonia. Experiments in the laboratory and on a large scale and experience at many sewage-disposal works have shown that, though the presence of the gas-works effluent in the sewage does throw an additional burden on the sewage-works, sewage containing as much as 0·5 per cent. of gas-effluent can usually be satisfactorily treated without causing an undue deterioration in the quality of the treated sewage. One important condition is that the gas-works effluent should be admitted to the sewers in controlled rates of flow, roughly in proportion to the flow of sewage, and not in flushes.

A vote of thanks was proposed by Mr. S. B. Donkin, President Inst. C.E., and seconded by Dr. William Cullen, President I. Chem. E. Eleven speakers took part in the oral discussion.

JOINT MEETING WITH THE BRITISH SECTION, SOCIÉTE
DES INGÉNIEURS CIVILS DE FRANCE AND THE
INSTITUTION OF STRUCTURAL ENGINEERS.

1 February, 1938.

SYDNEY BRYAN DONKIN, President Inst. C.E.,
in the Chair, supported by

WILLIAM THOMSON HALCROW, President, British Section,
Société des Ingénieurs Civils de France, and

Professor JOSEPH HUSBAND, M.Eng., President Inst. Struct. E.

" Dunkirk Harbour Extension Works."

By Monsieur L. P. BRICE.

(*Abridged.*)[1]

Monsieur L. P. BRICE referred to Dunkirk Harbour as one of the
principal transit points between England and France, and stated
that its history went back as far as the very origins of the history of
France.   During the nineteenth century the traffic of the Port
showed a steady increase and constant improvements were effected.
Extension and equipment works had also to be undertaken during
the War, but it was immediately after the War that the present ex-
tension programme was drawn up.   That included :—

(1) extending the present eastern breakwater by means of an
open-work breakwater, 700 metres long ;
(2) building a new solid western breakwater, 750 metres long,
laid out at an angle of about 98 degrees with the eastern
breakwater and leaving an entrance of 210 metres between
the ends of the two breakwaters ;
(3) connecting the western breakwater to the land by means of
a sea-wall ;
(4) building a sea-wall south of the new lower harbour ;
(5) building a marine lock ;
(6) building many miscellaneous works, such as a train jetty, a
berthing jetty, a new berth for the ferry-boat, a light-
house, etc.

Monsieur Brice stated that the eastern breakwater was composed essentially of open work similar to the Calais western breakwater. The sub-structure was formed of reinforced-concrete caissons sunk by compressed air, above which was a solid block with a facing of artificial stone and the inside filled with concrete. On it was super-imposed the upper structure made of reinforced-concrete truss girders, 7 metres high, and 3·05 metres wide at the top. The weight of the caissons varied from 300 to 400 tons. Each caisson was built up in thin reinforced concrete on a fixed platform alongside the channel at the shore end of the breakwater, and had an inner work-chamber. In launching the caisson particular care was taken in securing the connexions between the work-chamber and the outside atmosphere by a suitable system of cocks, so as to ensure that the caisson was sufficiently buoyant. After the caisson had been towed by tugs to its site, it was fastened to the service bridge, from which it was immediately ballasted, as the sea-level went down. The caisson grounded at low tide, and sufficient concrete was poured in to ensure that it did not float again. The ground underneath was disintegrated by injection-nozzles, which were connected to a pump working at a mean effective pressure of 6 kilograms per square centimetre. The loosened ground was removed by means of a Mammouth pump operated by compressed air ; that method was found to give complete satisfaction in sand and mud, and enabled the work to be carried out with the minimum employment of labour. The work-chamber was filled with concrete as soon as the caisson had reached the required depth, and then the whole caisson was concreted. The solid block above the caisson was completed, holes being left for the bottom legs of the reinforced-concrete beams forming the upper structure. The beams were made up on land, and after being brought to the site by barges, were lowered into place, and the whole completed.

Owing to the unsatisfactory nature of the ground, the western breakwater had to be of solid construction. For about 235 metres from the shore end, the breakwater was of the vertical type with the foundations consisting of a solid block of concrete poured between two curtains of sheet-piling 6·50 metres apart, and sub-divided into compartments. The upper structure included a solid mass of concrete, 5·70 metres wide at the bottom, with tapering (1 in 10) facings of artificial stone. Beyond the first 235 metres the breakwater was of the ordinary type with foundations resting on a bed of stone pitching. It included an upper structure in masonry and concrete resting on artificial blocks fastened on a stone-pitching core which rested on a fascine raft. Details were given of the methods employed in making the fascines and in placing the pitched stones,

and of the making and sinking of the artificial blocks.   The
pitching stone consisted of large-size pieces from 1,200 to 5,000
kilograms, and medium pieces from 100 to 1,200 kilograms, which
constituted the greater part of the core.

The marine lock was built to join the new lower harbour formed
by the eastern and western breakwaters to the upper existing
harbour.   The total length was 325 metres with an effective length
of 280 metres, the width being 42 metres in the dock and 40 metres
at the gates.   The lock was built on the site of the old beach, and,
in order to isolate the excavation from the sea, a cofferdam formed
up of two sheet-piling curtains, 8 metres apart, was built.   The
earthwork consisted of three stages, dredging, lowering of the water-
level, and dry earthwork.   The dredging was done by suction-
dredger, whilst the lowering of the water-level was effected through
a continuous belt of sixty filtration-wells connected by a pressure-
main.   Exhaust-pumps for operation under water were placed in
each well.   All the operating and controlling arrangements were
grouped in six pumping stations sited along the perimeter of the
excavation.   After 5 months' pumping it was possible to start on
the dry earthwork.   The earth was removed by caterpillar cranes
with buckets and carried by a line-conveyor to be dumped on the
level ground near the lock.   After completion of the work, the earth
was used to refill behind the side walls of the lock.

Many problems arose in connexion with the driving of the piles
as, owing to the very fine sand, direct driving was rendered very
difficult and often impossible.   A high-pressure steam-driven pump
was utilized to inject water to loosen the ground, with the result
that the sinking of the sheet-piling in certain cases was obtained
after 2,000 blows whereas 13,000 blows were required when the ground
could not be loosened.   In certain cases when very heavy sheet-
piling was being driven, it was necessary to loosen the ground on
both sides by injecting water under pressure.   It was found that
short-blow hammers were satisfactory for driving through sand but
unsatisfactory for driving through clay.

The side walls of the lock were built up of caisson-type sheet
piling, and were surmounted by crown walls of reinforced concrete.
The sheet piling was anchored by three sets of steel ties.   The
bottom of the lock was not concreted, except near both heads.   The
lock-heads consisted of concrete masses founded on a reinforced-
concrete general culvert bottom, protected against undermining
by a curtain of sheet-piling.   The head of the lock had two gate-
recesses, into the first of which the dock-gate slid.   The other one,
fitted up as a graving dock, included a spare gate.

The sea-walls were built up of concrete slabs moulded in advance,

resting on a stratum of marl and gravel earth. The slabs were anchored by means of reinforced-concrete piles. Vibration was employed in the manufacture of the slabs, beams and piles to ensure that the concrete was compact. Details were given of the methods of carrying out the work of laying the slabs, etc.

The Paper was illustrated by a number of lantern-slides and by films.

A vote of thanks was proposed by Mr. S. B. Donkin, President Inst. C.E., seconded by Mr. W. T. Halcrow, President, British Section, Société des Ingénieurs Civils de France, and supported by Professor Joseph Husband, President Inst. Struct. E.   The Meeting concluded with a short discussion.

Paper No. 5126.

"Some Experiments on Locomotive Springs, with Reference to Bridge Impact-Allowances."

By WILLIAM EDWARD GELSON, M.Sc. (Eng.), Assoc. M. Inst. C.E.

(Ordered by the Council to be published with written discussion.)[1]

TABLE OF CONTENTS.

INTRODUCTION.

THIS investigation was undertaken for the purpose of ascertaining the character and amount of the damping forces developed in the laminated springs of a locomotive when there is relative vertical motion between its sprung and unsprung portions, such as may occur when the locomotive is crossing a bridge.

SPECIAL TESTING APPARATUS.

A machine for applying measured pulsating and suddenly-applied loads to the springs was designed by the Author. It was constructed by the North Western Railway of India and was erected on railway premises at Lahore.

A general-arrangement drawing of this machine set up for pulsating-load tests is given in Figs. 1, Plate 1. Bending strain equivalent to the designed static weight on the locomotive-spring is applied by flexure of the girder G, the load being applied to the specimen through four hangers L, the housing H, and the coil spring, which has a stiffness-constant of the same order as that of the specimens. The system was stressed as described above by applying a known

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[1] Correspondence on this Paper can be accepted until the 15th June 1938, and will be published in the Institution Journal for October, 1938.—SEC. INST. C.E.

desired dead load through the tops of the hangers L, inserting cotters and suitable packings at the lower eyes of the hangers, and then removing the dead load.

The hangers are slotted to admit $\frac{1}{2}$-inch diameter rods welded to them near their lower ends, and there is a small gap between the upper ends of the slots and the tops of the $\frac{1}{2}$-inch rods. Calibration-curves were first obtained for each hanger, in which the amount of gap is plotted against the applied dead load in the hanger. This provides a ready method at any time for checking the dead load on the specimen.

A shaft S, to which various out-of-balance masses can be attached, runs in self-aligning " Skefko " ball bearings at each end. One bearing is located in the housing H, which has a degree of freedom in the vertical plane, and the other bearing J is carried in the main framework and fixes the position (but not the direction) of the shaft-axis at this point. The shaft is driven by a 20-h.p. electric motor through a variable-speed belt-and-chain drive, and in normal operation forced oscillation in the vertical plane about J as centre is imparted to the coil spring, the housing H, and the specimen and the shaft. A range of balance-weights is provided so that springs for axle-loads of up to 22·5 tons can be tested under their normal static load and under a pulsating load between limits of $\pm$ 4 tons at 1 revolution per second, and between $\pm$ 7 tons at 5 revolutions per second.

The machine described above was re-arranged as illustrated in Figs. 2, Plate 1, to enable the spring specimens to be tested under suddenly-applied loads. The main shaft S of the machine (Figs. 1, Plate 1) was removed, and a cradle loaded with kentledge was supported on jacks at B and on the two rollers at C, Figs. 2, Plate 1. The laminated-spring specimen was then placed in the machine upside down, supported at its ends on rollers by the girder G. The jacks B were then lowered until a small proportion of the cradle-weight was taken by the spring specimen through a spherical bearing, the packings P, and the spring-buckle. A lever L was then erected so that its nose was just bearing under the spring-buckle. The jacks B were then lowered out and removed. The trip-device T shown in the enlarged detail was then operated, and a record was taken of the spring-oscillation.

Suddenly-Applied Load Tests.—Suddenly-applied load tests were carried out with various weights of kentledge, and the effective combined mass of cradle and kentledge was determined in each case by inserting the coil spring shown in Figs. 1, Plate 1, and suitable packings between the housing D (Figs. 2, Plate 1) and the girder G instead of the specimen S (the lever L being removed), and measuring

the period of the swing of the cradle about the rollers C when a light vertical blow was delivered to the system. If f denotes the (undamped) natural frequency of swing of the system about C, the effective mass of the cradle and the kentledge $= \dfrac{K_c l^2}{4\pi^2 f^2}$, where K_c denotes the stiffness-constant of the coil spring, and l denotes the dimension (Figs. 2, Plate 1) 14 feet 8 inches.

Static Bending Tests.—Static bending tests were performed on each specimen in a 50-ton single-lever testing machine.

MEASURING INSTRUMENTS.

The oscillation of the spring specimens, both in suddenly-applied and in pulsating-load tests, was recorded by a Fereday optical bridge-deflectometer, fitted with a magnetically-operated time-marking mirror in circuit with a contact on the rotating shaft of the testing machine for use in pulsating-load tests to give the phase-relation between the deflexion and the pulsating force.

The small oscillations of the coil spring in tests to measure the effective mass of the cradle and kentledge (for suddenly-applied load tests only) were recorded by a Fereday-Palmer bridge-stress recorder connected between the girder G and the housing H.

Test for uniformity of angular speed of the shaft and balance-weights were made at intervals during the pulsating-load tests by providing incandescent electric bulbs attached to an arm secured to the end J of the rotating shaft. A cinematograph-camera taking 300 pictures per second and fixed with its optical axis along the main-shaft axis was used to record the position of the rotating bulbs in relation to four stationary bulbs forming axes of co-ordinates.

EXPERIMENTAL WORK.

The experimental work comprised static-load–deflexion, free-oscillation, and forced-oscillation tests, and the results of these are summarized below in the order in which the work was done.

Static-Load–Deflexion Tests.—Detail drawings of the three types of springs tested are included in *Figs. 3* (p. 298).

Six new springs, and six others of each type taken from locomotives in service, for driving and coupled axles carrying 20, 16 and 12 tons respectively (thirty-six springs in all) were selected for load–deflexion tests in a 50-ton Buckton single-lever testing machine. Two cycles from no load to 25 per cent. above nominal load and back to zero were performed in each case. Two each of the three types were further tested for deflexion-cycles of $\pm$ 0·1 inch, $\pm$ 0·3 inch, and $\pm$ 0·5 inch from normal-load deflexion.
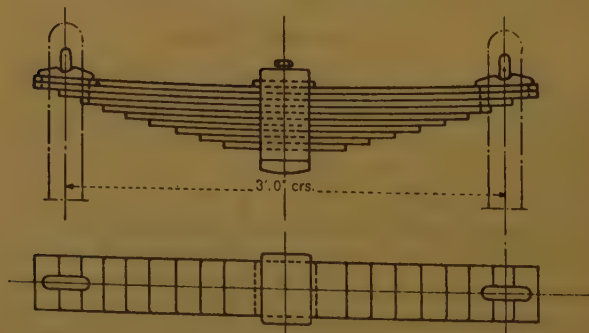
Figs. 3.
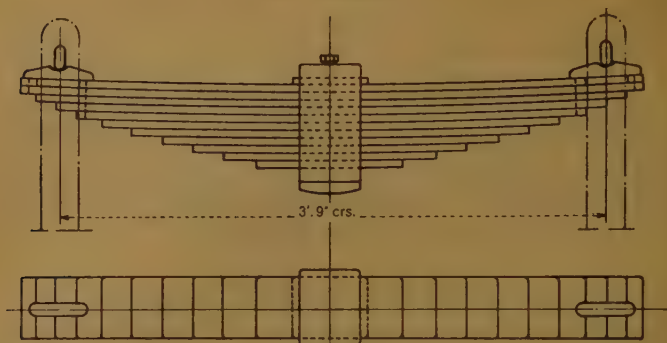
20-TON AXLE-LOAD.
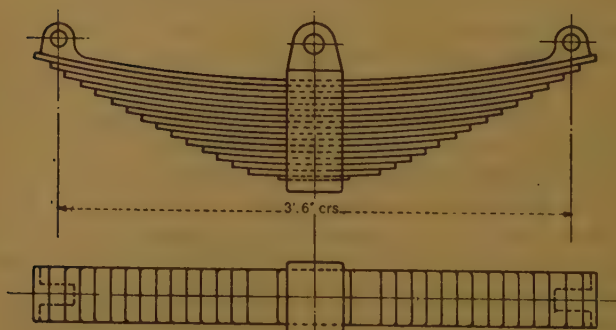
16-TON AXLE-LOAD.

12-TON AXLE-LOAD.

Scale: one-sixteenth full size.

Inches 12   9   6   3   0   1 foot

DETAILS OF SPRINGS TESTED.

The diagrams obtained from the new springs were similar to those from used springs, but the friction at normal load was always greater in the used springs, being on the average 19 per cent., 28 per cent., and 61 per cent. greater for springs for 20-, 16-, and 12-ton axle-loads respectively. The static-load–deflexion tests have furnished the data from which the curves in *Figs. 4* (p. 300) have been prepared. These are average curves of load and deflexion for ranges of $\pm$ 0·1 inch, $\pm$ 0·3 inch, and $\pm$ 0·5 inch from normal-load deflexion. The friction-force appears to be roughly proportional to the designed axle-load. It is found that the work done per cycle in overcoming friction varies in direct proportion to the range of deflexion, but it is zero for small deflexions up to a critical range of about $\pm$ 0·035 inch for each type of spring, and the stiffness-constant is much greater than for deflexions above this critical range.

Free Oscillations Produced by Suddenly-Applied Loads.—Six springs of each type were taken from those which had been statically tested, and kentledge was added to the cradle so as to impose dead loads of about 130 per cent., 100 per cent., and 60 per cent. of the designed static load in each case; these loads were suddenly applied twice to each specimen, and the free oscillations were recorded.

Two typical free-oscillation records are reproduced in *Fig. 5* (p. 301). Curve A is typical for an " XC " spring (20-ton axle-load) which exhibited $\pm$ 1·57 ton friction in the static tests. Curve B is typical for an " XA " spring (12-ton axle-load), which showed only $\pm$ 0·92 ton friction in the static tests.

In all tests with a suddenly-applied load the spring commences to deflect at a rate corresponding to its higher spring-constant; that is, the friction is not overcome until appreciable deflexion has occurred. The spring then executes damped free oscillations in accordance with the laws of solid friction, having a constant mean value but varying in proportion to the deflexion. At this stage, there is evidence that the interval from the first maximum to the second is somewhat briefer than that from the second to the third, and that that from the third to the fourth is somewhat briefer than that from the fourth to the fifth. When the oscillations have died down to the critical range below which the friction is not broken down, the system finally executes oscillations damped by fluid friction prescribed by the higher spring-constant. Due to a complication which was unforeseen, on account of the loaded cradle having subsequently been found to have a fundamental mode of oscillation of its own in flexure of about 7 periods per second, exact theoretical curves of free oscillation were not constructed for comparison with the records, which are regarded as qualitative. They serve to indicate that the spring-friction is of the " solid " type. This

Figs. 4.
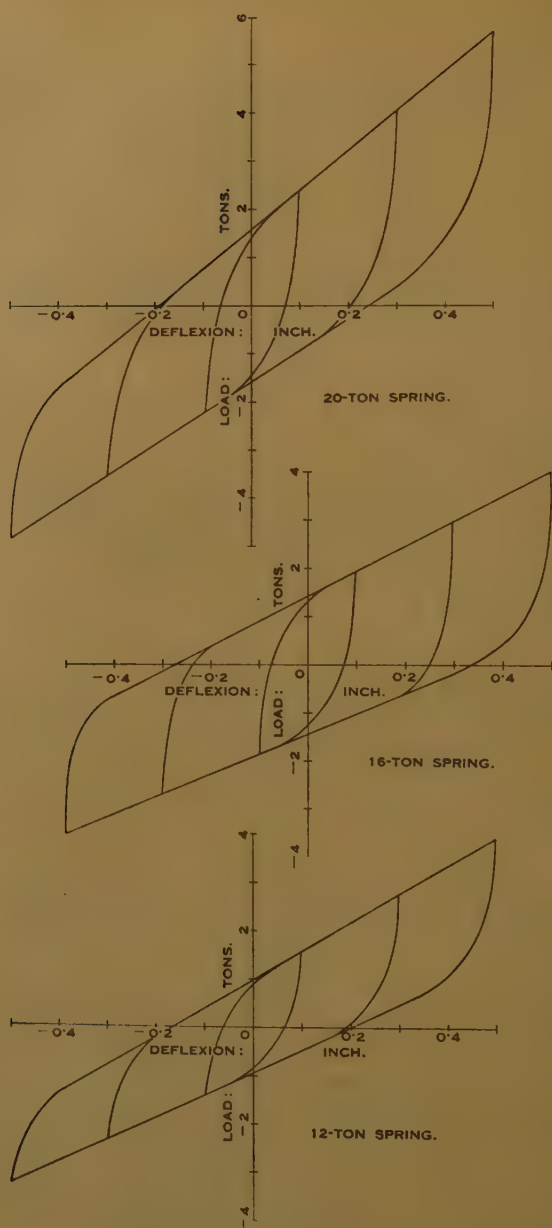
STATIC-LOAD–DEFLEXION TESTS.

difficulty could have been overcome by stiffening-up the cradle, but as the main object of this investigation is the study of forced oscillations the expense which would have been involved was never incurred.

Forced Oscillation Produced by Rotating.—In one series of tests, six of each of the three types of springs were subjected to loads of (1 ± 0.4) multiplied by the normal static load at 3, 4 and 5 revolutions

Fig. 5.
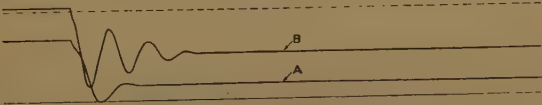
FREE-OSCILLATION RECORDS.

per second. In the other series three springs of each type were tested at a range of speeds of from 1 to 5 revolutions per second, using the same balance-weights selected to give a large force in each case at 5 revolutions per second for each speed. In all these tests the records showed that the instants of maximum and minimum pulsating load coincided with maximum and minimum deflexion, and this led the Author to think that the springs were damped by solid friction.

CALCULATION OF PULSATING FORCE.

If the centres of the balance-weights and shaft-axis lie in the same horizontal plane at time t_1, then the load on the spring at any time t is

$$\tau . \frac{M}{l} \sin \omega(t - t_1) \text{ approximately} \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

where τ is a magnification-factor which depends upon how nearly the frequency of pulsation agrees with the mean natural frequency of the system. (In all cases τ is a small fraction greater than unity.) M denotes the maximum value in tons-feet of the rocking moment due to out-of-balance masses in a vertical plane about the fixed bearing J (Figs. 1, Plate 1) ; l denotes the dimension 7.71 feet $= 7$ feet $8\frac{1}{2}$ inches (Figs. 2, Plate 1) ; and ω denotes the angular velocity of the shaft in radians per second.

The value of τ is given by $\tau = \dfrac{\dfrac{Ml}{I}}{\left(\dfrac{K_c + K}{I}\right)l^2 - \omega^2}$,

where I denotes the average moment of inertia in ton-foot-second units of the oscillating mass (namely, the shaft, the out-of-balance weights, the housing and the effective portion of laminated and coil

20

springs): K_c and K denote the stiffnesses of coil and laminated springs respectively, in tons per foot of deflexion. I fluctuates between two limits in each cycle, and it was determined by direct experiment.

A correction to equation (1) to allow for the motion of vibration should be made, but it can be shown that the error on this account is not greater than 4 per cent. in extreme cases.

Other corrections are of a still smaller order, and no corrections to equation (1) have been made in the construction of the theoretical

Figs. 6.
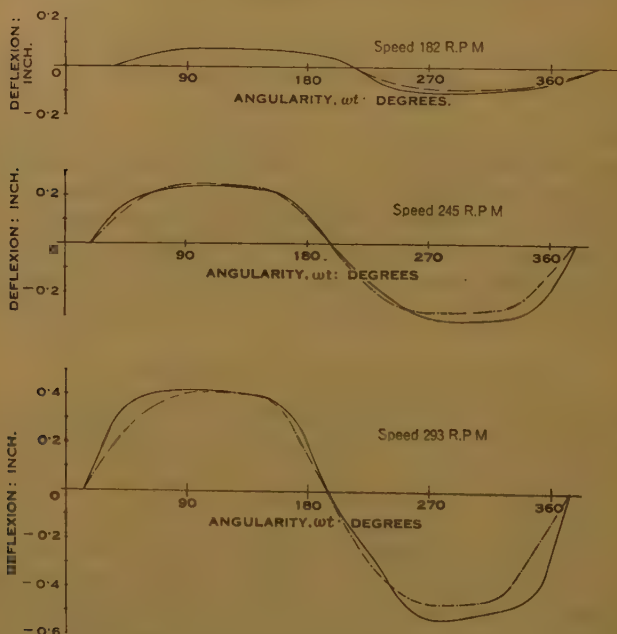
DYNAMIC DEFLEXION-CURVES.

curves, typical examples of which are given in *Figs. 6* for " XC " springs in chain-dotted lines, together with the actual observed deflexions in full lines, to a base of time. The theoretical curves have been derived from the deflexions in *Figs. 4* corresponding to the loads given by equation (1).

The agreement is in all cases probably as good as could be expected with the measuring apparatus used, both when the friction between the leaves has and has not been overcome, and demonstrates that the deflexion of a locomotive laminated spring under a known pulsating force can be determined directly from the static-load-deflexion diagram.

CONCLUSIONS.

Curves were prepared from *Figs. 4* showing frictional resistance to a base of time for each type of spring for spring-movements of

Fig. 7.

FRICTION RESISTANCE, F: TONS.

Class " XC ": 20-ton axle-load

Class " HG/S ": 16-ton axle-load

Class " XA ": 12-ton axle-load

AMPLITUDE OF SPRING-MOVEMENT: INCH.

Fig. 8.

ANGLE OF LEAD: RADIANS.

- AMPLITUDE OF SPRING-MOVEMENT: INCH.
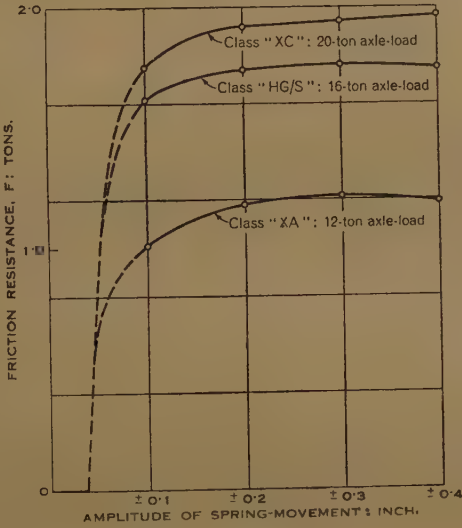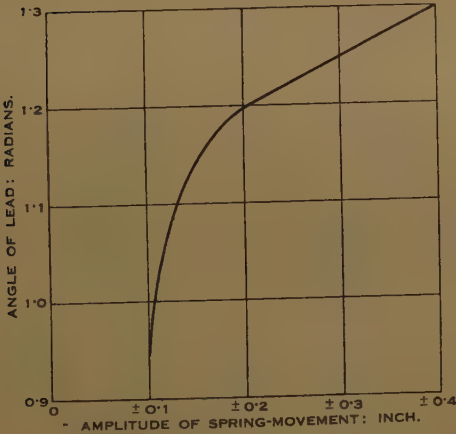
$\pm$ 0·1 inch, $\pm$ 0·2 inch, $\pm$ 0·3 inch, and $\pm$ 0·4 inch. From these the fundamental harmonic components have been evaluated for each deflexion-range, and the results are summarized in curves as

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Fig. 7 (p. 303), which shows the value of the friction of one spring to a base of spring-displacement corresponding to the factor $\frac{4}{n}$ given in the Report of the Bridge Stress Committee.[1]   The curve of *Fig.* 8 (p. 303) gives, to a base of spring-displacement, the angle by which the fundamental harmonic component of the friction-force leads the spring-displacement, and applies to each type of spring.

ACKNOWLEDGEMENTS.

The Paper is accompanied by twenty-two sheets of diagrams and a series of films of instrument-records, from some of which Plate 1 and the Figures in the text have been prepared, and by five photographs.

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[1] London, 1928.

SOME EXPERIMENTS ON LOCOMOTIVE SPRINGS, WITH REFERENCE TO BRIDGE IMPACT-ALLOWANCES.

PLATE 1.
EXPERIMENTS ON LOCOMOTIVE SPRINGS.

FIGS: 1.

FIGS: 2.

SECTIONAL ELEVATION ON AA.

FRONT ELEVATION (FRAMING ON SPRING-SIDE OMITTED).

CROSS SECTION.

ELEVATION.

SECTION ON AA.

PLAN.

PLAN.

ELEVATION.

SECTIONAL PLAN.

DETAILS OF SPRING-SIDE BEARING H.

Scale for details : 1 inch = 2 feet

SIDE ELEVATION.

FRONT ELEVATION.

DETAILS OF SPRING SUSPENSION.

SECTION.

ELEVATION.

SECTION.

DETAILS OF BEARING J.

Scale for details : one-sixteenth full size.

ELEVATION.

SIDE.

PLAN.

TRIPPING DEVICE.

EXPERIMENTAL APPARATUS.

RE-ARRANGEMENT OF EXPERIMENTAL APPARATUS.

WILLIAM CLOWES & SONS, LIMITED: LONDON.

The Institution of Civil Engineers.   Journal.   March, 1938.

W. E. GELSON.

Paper No. 5117.

"Sea-Defences at Admiralty Pier, Dover."

By JOHN WILLIAM SUTTON, Assoc. M. Inst. C.E.

(Ordered by the Council to be published with written discussion.)[1]

TABLE OF CONTENTS.

INTRODUCTION.

IN recent years the condition of the masonry on the face at the inshore end of the Admiralty pier, Dover, has caused some concern owing to damage by sea-action. This damage, which is now confined to the west side of the pier, has mainly taken the form of loosened facing blocks, weighing on an average 3 tons each, being forced out of position by air and water pressure generated behind the face-work during rough seas. Hitherto, when the blocks have been recoverable they have been replaced and jointed, or alternatively the cavities formed have been shuttered and filled with mass concrete. Until 1912, air trapped in the heart of the pier during rough weather found a vent in the opposite face of the pier to the running swell, but from that date, when the widening of the Admiralty pier was completed, the west-side face-work has needed careful watching. In addition, the extension of the pier in a nearly easterly direction has increased the run of the sea along the wall, so that with any wind the sea-conditions in this area are uncomfortable. The repair-methods employed as set out above gave good results, except that in some instances the trouble appeared to be transferred to a near or adjacent block, indicating that a void existed behind the face-work. In general, the dislodgements occurred at or just above the level of L.W.O.S.T.

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[1] Correspondence on this Paper can be accepted until the 15th June 1938, and will be published in the Institution Journal for October, 1938.— SEC. INST. C.E.

STORM-DAMAGE.

At the inner end of the pier the face-courses are in Bramley fa
with thinly-bedded blocks in course formation, the foundatio
course being 3 feet 2 inches deep and the top course 1 foot 3 inche
deep. No other security than the bedding was provided, and th
present general condition of the masonry is remarkable after 8
years' constant exposure to extremely bad conditions.

Following the very heavy storm of the 17th September, 1935, t
which this work was fully exposed, an inspection disclosed that
settlement had taken place at the inshore end for a distance o
40 feet, commencing 20 feet from the shore (L.W.O.S.T.) and extend
ing seawards. Ten courses, 25 feet in height, were affected, th

Fig. 1.
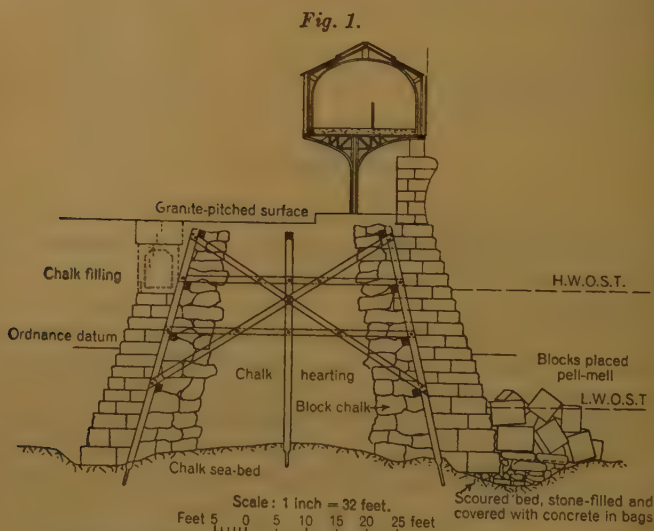
CROSS SECTION OF PIER AT SITE OF INSHORE SETTLEMENT.

settlement at approximately the centre being $2\frac{1}{2}$ inches. A second
and smaller settlement had also occurred at a point 100 feet from
the shore. Considerable blowing at the joints was set up when a
swell ran along the pier, indicating the existence of hollows behind
the face-work. At each local area a face-block had been forced
away, and an examination in the cavities formed revealed that the
backing to the face-work was suffering some displacement and that
the chalk hearting of the pier was being broached by sea-action. An
underwater inspection showed the cause of the settlement, which
was erosion of the chalk bed in front of and underneath the founda-
tion course, extending 300 feet seaward; in some parts hollows

feet deep had been formed. *Fig. 1* is a cross section of the pier in the area under review, and shows the scouring of the sea-bed.

REPAIRS NECESSARY.

Repair-work was divided into two sections :—

(1) The formation of an apron in the scoured sea-bed and the provision of protection to prevent further erosive action.

(2) Making good the masonry face of the pier, and filling as far as possible the voids behind the blocks.

The first part could be carried out only when sea-conditions were favourable, such conditions being only likely to occur for short periods during the summer. The second part could be carried on during fair or fine weather, and would have to be done mainly at low tide, except that the sealing of the underwater joints would have to be carried out by divers. Preparations were therefore made with a view to pressing forward with the work as soon as favourable conditions permitted. Actually from the time the settlement took place 10 months elapsed before continuous fine weather prevailed.

METHOD OF MAKING REPAIRS.

Formation of Apron.—The eroded area under the bed course was first filled with concrete lumps, granite blocks and concrete in bags in sizes capable of being handled and placed under water by divers ; the scoured trench in front of the foot of the wall was then filled with a double layer of concrete in bags, each weighing nearly 2 cwt., and placed as uniformly as possible, thus forming a toe-apron. A total of 40 tons of material was thus deposited.

Following this a pell-mell-block wave-breaker was formed for a length of 100 feet at the shore end of the work to hold and protect the apron, and to provide additional defence for the face of the wall against sea-action. The need for this was emphasized when, during one day of bad weather shortly after the commencement of the work, a 20-foot length of the toe-apron close to the shore was torn away. For the pell-mell formation large dressed-granite blocks with an average weight of 10 tons were conveyed to the site and deposited. By reason of their cubic shape the blocks were roughly fitted together to prevent dispersal by rough seas until such time as shingle collected and provided additional bedding. The trend of the beach here is from west to east. Before the weather caused a suspension of operations sixty blocks had been placed. There is every indication that this formation has effectually provided a sound defence against under-toe scouring, for the unprotected apron

from 100 to 300 feet seaward is still intact. One addition of six
blocks has been made to this pell-mell formation. Further additions
are contemplated.

Before depositing these blocks all joints in the masonry below

Fig. 2.
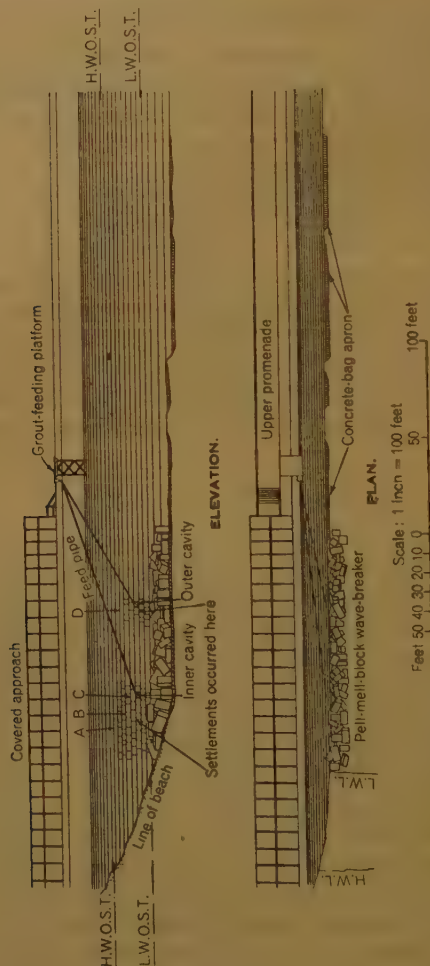
METHOD OF REPAIRING MASONRY FACE OF PIER.

water were caulked, whilst above the level of L.W.O.S.T. all joints
were sealed with Portland cement in both affected areas, in pre-
paration for the second part of the work.

Repairs to Masonry Face of Pier.—It was decided to deal with this
part of the work from the pier-parapet, as the risk of working from
a barge was too great owing to possible sudden changes in the sea-

conditions. The nearest point to the work was 40 feet above it and about 120 feet away. A platform and a 4-inch diameter feed-pipe were provided as shown in *Figs. 2*. Grout-holes, 5 inches in diameter, were drilled in the face-work, so arranged as to strike the joint between the block immediately below and the backing course. These are indicated in *Figs. 2* at points A, B, C, and D.

The inshore area was dealt with first, the cavity being shuttered and fed with cement-grout, together with a small quantity of concrete when the work was nearing completion. The operation was carried out during one low-tide period. Following this, for 3 days during the morning low tides, holes A, B, and C were fed with cement-grout under gravity. Feed-holes A and B were both fed successfully, but feed-hole C soon became fully charged. In all 16 tons of material was used in this area. A similar procedure was adopted with the cavity and feed-hole D of the outer area, the work being completed in three low-tide periods. In this area 10 tons of material was used.

No estimate could be formed of the quantity of cement-grout that would be required, owing to an almost complete lack of knowledge of the possible voids existing, and sufficient material for each working period was therefore transported to the site during the high-water periods. Fortunately, the weather remained good during the time that this work was being carried out; the operations covered two ranges of spring tides, and was carried out at the same time as the placing of the pell-mell-block sea-defence. Since the end of September, 1936, a few additional blocks have been placed and pointing at the joints made when quiet weather-conditions have occurred.

Conclusion.

Inspections indicate that there are no serious voids behind the Bramley-fall facing, as there is now no sign of blowing following sea-swell during bad weather, which is the chief symptom of this trouble. Several joints have been raked out to allow these examinations to be made, and in most cases the new cement-grout has been uncovered.

The Paper is accompanied by two sheets of drawings, from which the Figures in the text have been prepared.

ENGINEERING RESEARCH.

DEPARTMENT OF SCIENTIFIC AND INDUSTRIAL RESEARCH.

REPORT FOR THE YEAR 1936–37.

According to the Report, the net expenditure for the year was approximately £582,000, an increase of £10,000 on that of the previous year. The aggregate sum subscribed by industry through the Research Associations has increased to nearly £250,000 and the Government grants-in-aid to approximately half this amount.

New laboratories have been erected for an extension of acoustics research at the National Physical Laboratory, and larger premises have been acquired near Covent Garden for the investigation of food-problems. Preparations are being made for the construction of two new wind-tunnels at the National Physical Laboratory.

The following notes are selected from those parts of the Report of engineering interest:—

The completion of 10 years' research into water-pollution has been made the occasion for a review of the work done. Since last year's Report it is noted that the survey of the river Mersey, which was undertaken in view of the fear of silting of the Mersey estuary as a result of the discharge thereinto of crude sewage, has just been completed. Further work has been done on base-exchange methods of purification, the exchange-properties of synthetic resins, lead contamination, dairy waste-waters, and the purification of sewage.

Certain activities of the Department have already been reviewed in the Journal in connexion with the Annual Reports on the work at the National Physical Laboratory,[1] on Building Research,[2] Forest Products Research,[3] Fuel Research,[4] and Road Research.[5]

In connexion with food investigation, the improvement of cold-storage methods of transport and the thermal properties of refrigerators have received attention. Metallurgical research has been carried out into the behaviour of metals at high temperatures, and mechanical tests, conducted in vacuo, have been carried out on

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[1] Journal Inst. C.E., vol. 6 (1936–37), p. 283. (June 1937.)
[2] —— vol. 6 (1936–37), p. 582. (October 1937.)
[3] —— vol. 7 (1937–38), p. 309. (December 1937.)
[4] —— vol. 7 (1937–38), p. 310. (December 1937.)
[5] —— vol. 6 (1936–37), p. 285. (June 1937.)

specimens of pure iron. The effect of the addition of carbon in various proportions has been studied. Work has been done on alloys of nickel, chromium and iron, on aluminium and magnesium alloys, and on copper steels. In chemical research a study has been made of the corrosion of metals, both in the atmosphere and immersed in fresh and sea water. The corrosion of steel and cast iron by sulphate-reducing bacteria is also being studied. Further work has been done on the hydrogenation of coal. A study has been made of the resistance of metals to the corrosive action of tar fractions. Radio research has included studies of the ionosphere and the propagation of waves, direction-finding, ultra-short waves, and atmospherics. On the subject of illumination research, the relative merits of different-coloured lights with particular reference to street lighting are being investigated. In connexion with the lubrication of journal bearings, incipient seizure has been studied ; experiments have also been carried out on rubber bearings with water lubrication, and on oscillating journals. Reports are also given on the researches into atmospheric pollution, furnace-design, X-ray analysis and gas-cylinders and containers.

Work in connexion with the British Cast Iron Research Association includes a study of graphite-refining, high-duty and alloy irons and vitreous enamelling of cast iron. Researches for the British Iron and Steel Federation include work on the blast-furnace, open-hearth furnace and the heterogeneity of ingots. The British Non-Ferrous Metals Research Association has investigated the melting and casting of metals, creep, and the corrosion of zinc coatings. The section of the Report dealing with the British Electrical and Allied Industries Research Association indicates progress on problems in connexion with dielectrics, earthing, capacity of cables, wear of overhead contact-wires, efficiency of steam-power plant, surge phenomena, transformer noise, safety problems and circuit-breaker research. The British Scientific Instruments Research Association has studied the durability of optical glasses, and tarnishing and corrosion in instruments. The difficulties of winter painting are receiving the attention of the Research Association of British Paint, Colour and Varnishing Manufacturers. In connexion with the Institution of Automobile Engineers Research and Standardisation Committee, the performance of engine-bearings, noise in motor vehicles, and the deep drawing of metals have been studied ; the research on cylinder-wear is now nearing completion. The effect of the greasing of belting leathers has been studied by the British Leather Manufacturers Research Association. The durability of rubber jointing and other ageing problems has received the attention of the British Rubber Manufacturers' Research Association. Research in connexion with the British Colliery Owners'

Research Association has included a study of the suppression of dust, underground illumination, and atmospheric conditions in deep mines.

RESEARCH IN ENGINEERING AT UNIVERSITY COLLEGE, SOUTHAMPTON.

Engineering research at University College, Southampton, which is being carried out under the direction of Professor T. R. Cave-Brown-Cave, C.B.E., is concerned with problems in connexion with the exhaust noise of internal-combustion engines and wind-tunnel research.

Exhaust Noise.

Research has been carried out during the last few years to investigate the problems connected with reducing the exhaust and other noises of internal-combustion engines, particularly those of motor vehicles. The exhaust noise contains a low-pitch component caused by the cylinder acting as a resonator while the valve is open, and a high-pitch component caused by the gas escaping at high pressure through the exhaust valve as it opens. The effectiveness of various types of silencer in suppressing each of these components has been investigated.[1] In addition to the exhaust noise, there is a large amount of sound emitted from the inlet port similar to the low-pitch component of the exhaust. Although this sound is not quite so great as the corresponding exhaust noise, its nature is such that a silencer of almost the same dimensions is required to suppress it satisfactorily. Investigation has also shown that the mechanical noises emitted by the engine are very largely due to vibration transmitted to the supporting framework and thence to large surfaces capable of transmitting them to the car.

Wind-Tunnel.

A 5-foot diameter open-jet wind-tunnel has been constructed on lines closely similar to those of one at the Royal Aircraft Establishment, Farnborough, giving a maximum air-speed of about 100 feet per second. The tunnel is of simple, inexpensive construction, yet the flow is remarkably steady and uniform across the jet. The investigations which have so far been carried out in the tunnel have mostly taken the form of exploring the nature of the air-flow over models of motor-cars, motor-buses and buildings. Preparations are being made for an investigation of the aerodynamics of sails.

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[1] Proc. Inst. Auto. Engineers, vol. 31, March, 1937, p. 783.

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